407 - 444

QUINNIPIAC RIVER BASIN
MERIDEN, CONNECTICUT

BRADLEY HUBBARD RESERVOIR DAM
CT 00132

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

SEPTEMBER, 1980

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

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19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,
Quinnipiac River Basin
Meriden, Conn.
Bradley Hubbard Reservoir Dam

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The project has a total length of 545 ft. consisting of 340 ft. masonry core with earth fill on the upstream and downstream sides, a 115 ft. long earth embankment at the right end of the dam, and section of concrete corewall at each end. It is classified as a high hazard, small size dam. The test flood range is from one-half to full Probable Maximum Flood.

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QUINNIPIAC RIVER BASIN
MERIDEN, CONNECTICUT

BRADLEY HUBBARD RESERVOIR DAM
CT 00132

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

SEPTEMBER, 1980

BRIEF ASSESSMENT

PHASE I INSPECTION REPORT

NATIONAL PROGRAM OF INSPECTION OF DAMS

Name of Dam:	BRADLEY HUBBARD RESERVOIR DAM
Inventory Number:	CT 00132
State Located:	CONNECTICUT
County Located:	NEW HAVEN
Stream:	HARBOR BROOK
Owner:	CITY OF MERIDEN
Date of Inspection:	MAY 12, 1980
Inspection Team:	PETER HEYNEN, P.E.
	DR. MURALI ATLURU, P.E.
	MIRON PETROVSKY
	JAY A. COSTELLO
	JEFFREY BORNE

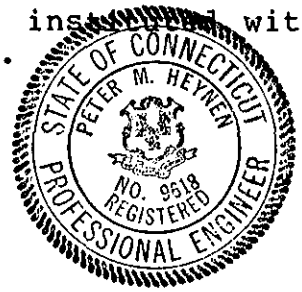
The project, built about 1891 has a total length of 545 feet consisting of a 340 foot masonry core with earth fill on the upstream and downstream sides, a 115 foot long earth embankment at the right end of the dam, and sections of concrete corewall at each end (See Sheet B-1). A 71 foot broad-crested masonry spillway is located at the central portion of the dam. The top of the dam (elevation 312.0) is 7 feet wide and 16.5 feet above the Harbor Brook streambed. The maximum storage capacity with the pond level to the top of the dam is approximately 216 acre-feet of water. A gatehouse, located upstream and adjacent to the right end of the spillway, contains two valves which regulate a 20 inch blowoff and a 12 inch supply main which once led to the Bradley and Hubbard Corp.

In accordance with the Army Corps of Engineer's Guidelines, Bradley Hubbard Reservoir Dam is classified as a high hazard, small size dam. The test flood range is from one-half to full Probable Maximum Flood (PMF). The selected test flood for Bradley Hubbard Reservoir Dam is equivalent to the PMF. Peak inflow to the reservoir at the test flood is 1500 cubic feet per second (cfs); peak outflow is 1325 cfs with the dam overtopped by 0.9 feet. The spillway capacity with the reservoir level to the top of the dam is 223 cfs, which is equivalent to 17% of the routed test flood outflow.

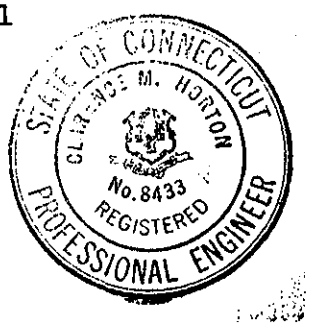
Based upon the visual inspection at the site and past performance, the project is judged to be in fair condition. There are items which require maintenance and/or evaluation, such as seepage, deteriorated masonry, the presence of animal burrows in the embankments, and the irregularites caused by erosion of the upstream and downstream embankments.

It is recommended that the owner retain the services of a registered professional engineer to analyze in more detail the adequacy of the existing project discharge and overtopping potential. Other items of importance are monitoring of seepage, repair of deteriorated masonry, repair of erosion and replacement of riprap at the right end of the upstream slope, filling of animal burrows, and the development of maintenance procedure and emergency action programs. Recommendations made by the engineer should be implemented by the owner.

The above recommendations and further remedial measures presented in Section 7 should be instituted within one year of the owner's receipt of this report.



Peter M. Heynen, P.E.
Project Manager - Geotechnical
Cahn Engineers, Inc.



C. Michael Horton, P.E.
Department Head
Cahn Engineers, Inc.

This Phase I Inspection Report on Bradley Hubbard Reservoir Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and are hereby submitted for approval.

ARAMAST MAHTESIAN, Member
Geotechnical Engineering Branch
Engineering Division

CARNEY M. TERZIAN, Member
Design Branch
Engineering Division

RICHARD DIBUONO, Chairman
Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

JOE B. FRYER
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspection. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam would necessarily represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions there of. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as neccessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

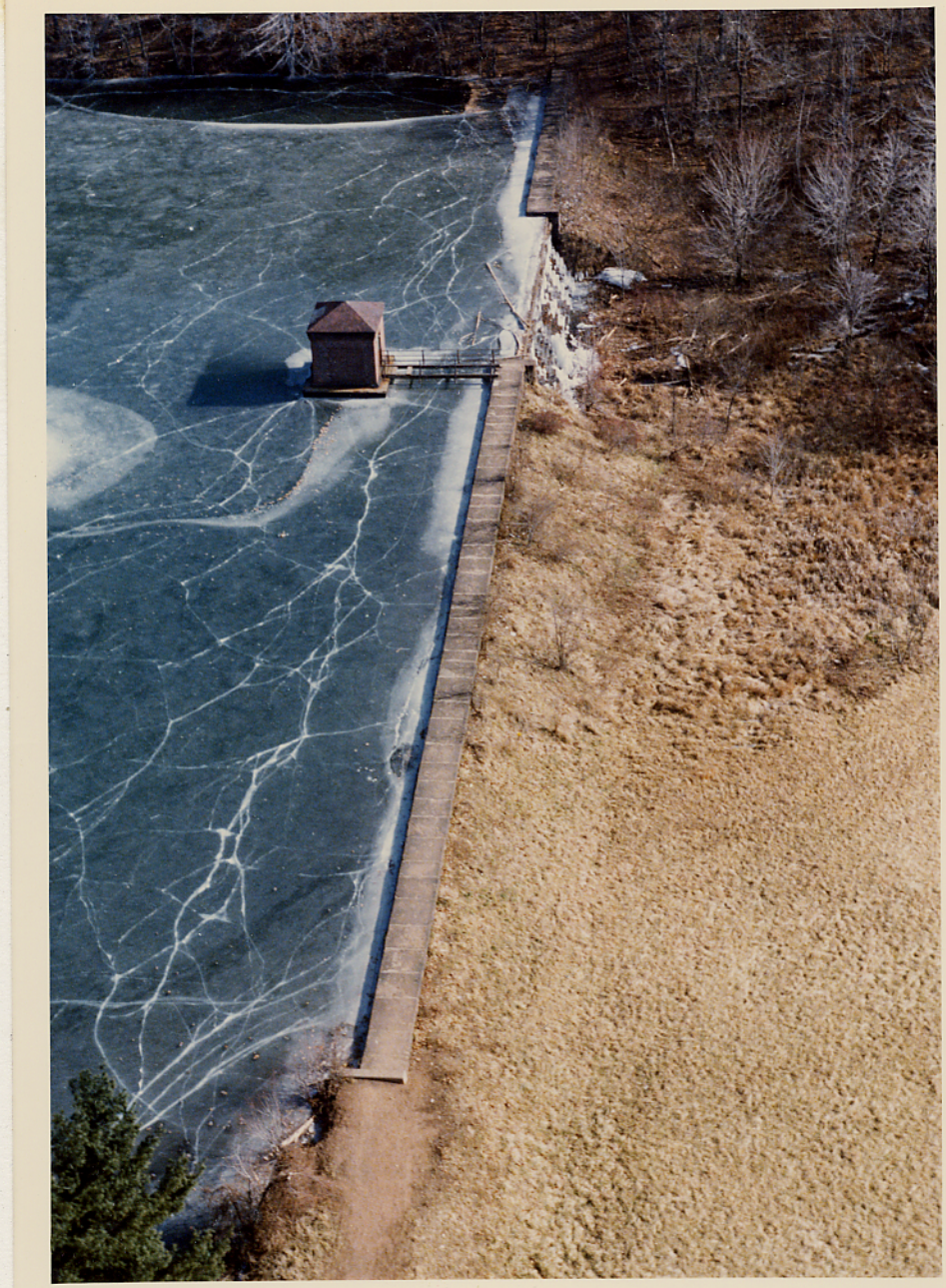
The Phase I Investigation does not include an assessment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings and other items which may be needed to minimize trespass and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

The information contained in this report is based on the limited investigation described above and is not warranted to indicate the actual condition of the dam. The integrity of the dam can only be determined by a means of a monitoring program and/or a detailed physical investigation. The accuracy of available data is assumed where not in obvious conflict with facts observable during the visual inspection.

TABLE OF CONTENTS

SECTION 7: ASSESSMENT, RECOMMENDATIONS & REMEDIAL MEASURES

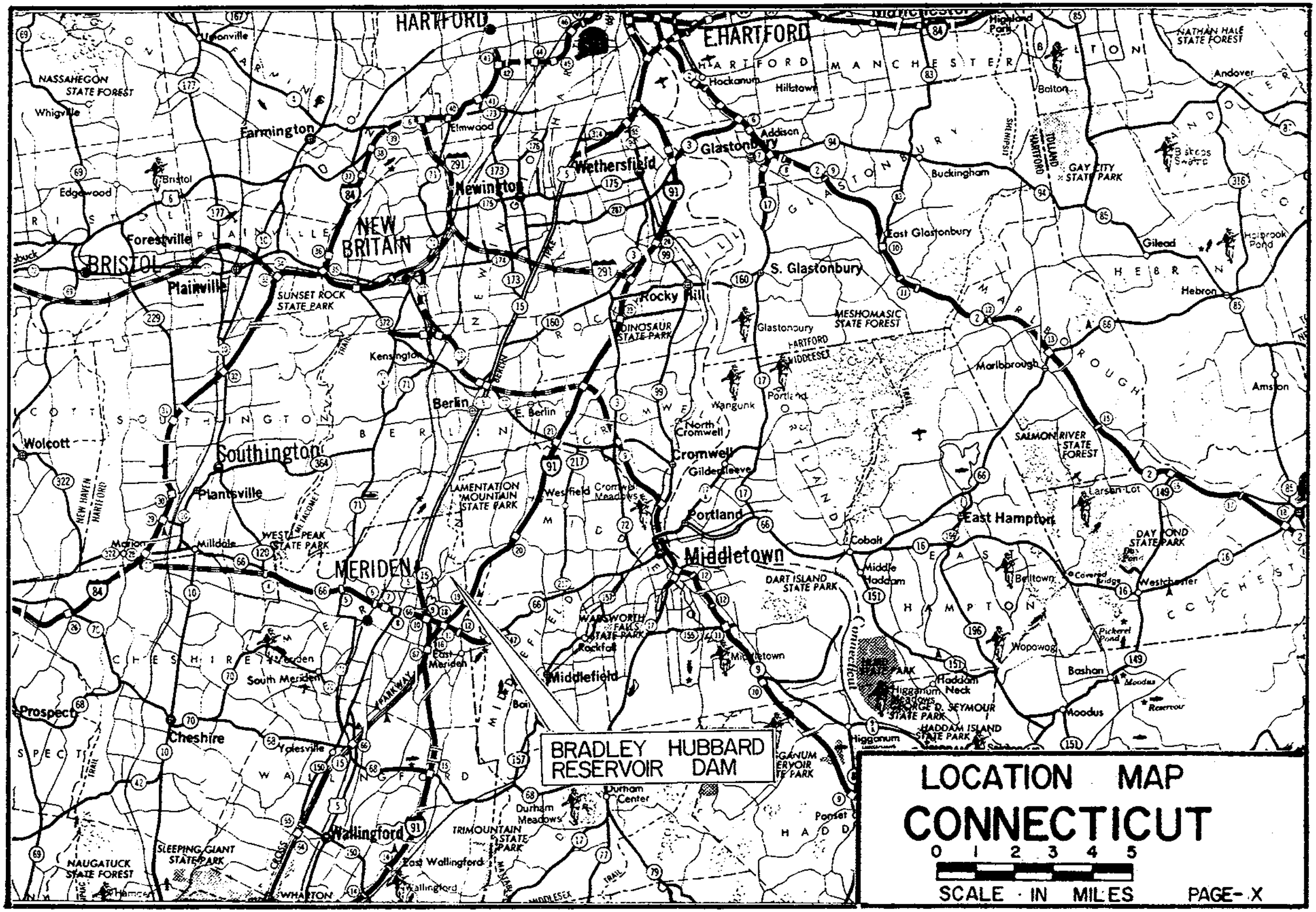
APPENDICES



OVERVIEW PHOTO

(February, 1980)

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Bradley Hubbard Res. Dam Harbor Brook Meriden, CT CE# 27 785 KE DATE Sept. '80 PAGE ix
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		


BRADLEY HUBBARD
RESERVOIR DAM
LOCATION MAP
CONNECTICUT
0 1 2 3 4 5
SCALE IN MILES
PAGE-X
HARTFORD
E.HARTFORD
NEW BRITAIN
BRISTOL
Forestville
Plainville
Farmington
Newington
Wethersfield
Glastonbury
S. Glastonbury
East Glastonbury
Rocky Hill
Kensington
Berlin
E. Berlin
Southington
Plantsville
Wolcott
Milldale
Marion
MERIDEN
East Meriden
South Meriden
Cheshire
Prospect
Yalesville
Wallingford
East Wallingford
Middlefield
Rockfall
Middletown
Portland
Cromwell
North Cromwell
Gildersleeve
Durham Center
Durham Meadows
East Hampton
Cobalt
Middle Haddam
Marlborough
Hebron
Gilead
Andover
Bolton
Buckingham
Elmwood
Hockanum
Hillstown
Addison
Whigville
Edgewood
Westfield
Westchester
Moodus
Bashan
Higganum
Haddam Neck
Amston
NASSAHEGON STATE FOREST
SUNSET ROCK STATE PARK
DINOSAUR STATE PARK
MESHOMASIC STATE FOREST
LAMENTATION MOUNTAIN STATE PARK
WEST PEAK STATE PARK
WADSWORTH FALLS STATE PARK
DART ISLAND STATE PARK
SALMON RIVER STATE FOREST
DAY POND STATE PARK
GAY CITY STATE PARK
NATHAN HALE STATE FOREST
SLEEPING GIANT STATE PARK
NAUGATUCK STATE FOREST
TRIMOUNTAIN STATE PARK
GEORGE D. SEYMOUR STATE PARK
HADDAM ISLAND STATE PARK

PHASE I INSPECTION REPORT

BRADLEY HUBBARD RESERVOIR DAM

SECTION I - PROJECT INFORMATION

1.1 GENERAL

a. Authority - Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Cahn Engineers, Inc. has been retained by the New England Division to inspect and report on selected dams in the State of Connecticut. Authorization and notice to proceed were issued to Cahn Engineers, Inc. under a letter of April 14, 1980 from William E. Hodgson, Jr. Colonel, Corps of Engineers. Contract No. DACW 33-80-C-0052 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection Program - The purposes of the program are to:

1. Perform technical inspection and evaluation of non-federal dams to identify conditions requiring correction in a timely manner by non-federal interests.

2. Encourage and prepare the States to quickly initiate effective dam inspection programs for non-federal dam.

3. To update, verify and complete the National Inventory of Dams.

c. Scope of Inspection Program - The scope of this Phase I inspection report includes:

1. Gathering, reviewing and presenting all available data as can be obtained from the owners, previous owners, the state and other associated parties.

2. A field inspection of the facility detailing the visual condition of the dam, embankments and appurtenant structures.

3. Computations concerning the hydraulics and hydrology of the facility and its relationship to the calculated flood through the existing spillway.

4. An assessment of the condition of the facility and corrective measures required.

It should be noted that this report passes judgment only on those factors of safety and stability which can be determined by a visual surface examination. The inspection is to identify those visually apparent features of the dam which evidence the need for corrective action and/or further study and investigation.

1.2 DESCRIPTION OF PROJECT

a. Location - The dam is located on Harbor Brook (Quinnipiac River Basin) in a suburban area of the City of Meriden, County of New Haven, State of Connecticut. The dam is shown on the Meriden USGS Quadrangle Map having coordinates latitude N41°33.5' and longitude W72°45.7'.

b. Description of Dam and Appurtenances - The dam totals more than 545 feet in length and consists of several sections; the original stone masonry section with earth fill added to the upstream and downstream sides, a 115± foot long earth embankment at the right end of the dam and a concrete corewall at each end of the dam extending into the natural earth abutment. The stone masonry section is 340 feet long and the earth embankment is 115± feet in length. The original masonry dam was raised 3.5 feet in 1912. At this time the earthfill was added at the upstream and downstream sides of the masonry and the earth embankment section was added to fill a low area at the right end of the dam. A concrete corewall was also added at each end of the dam. At the right end, the corewall abuts the original masonry and extends through the earth embankment to 175 feet from the masonry core. At the left end, a 30 foot section extends into the earth abutment and abuts the original masonry (See Sheet B-1). Raising the original dam consisted of removing the cap stones and placing a 3.5 foot thick section of concrete on the dam and replacing the cap stones, raising the dam 3.5 feet. (See Section B-B, Sheet B-1). The top of the masonry coping (elevation 312.0) is 7.0 feet wide, 1.0 foot above the spillway crest and 16.5 feet above the streambed at the toe of the dam. The top of the earth embankment section is approximately 15 feet wide and at elevation 313.0. The concrete corewall at the right end is 5.0 feet thick and tapers to 2.0 feet thick at the top, which is 1.0 feet below the top of the embankment (See Sheet B-1). The concrete corewall extension at the left end is approximately 5.0 feet thick.

The earth fill at the upstream side of the masonry is inclined at 2.5 horizontal to 1 vertical and is overlain by a rock fill which is inclined at 1.5 horizontal to 1 vertical and extends to 2± feet from the top of the masonry. A 2.0 foot thick section of concrete extends along the entire length of the upstream face of the masonry core and was placed at the time of the reconstruction (See Section B-B, Sheet B-1). The earthfill on the downstream side of the masonry core is inclined at 2.0 horizontal to 1 vertical and has a grass cover.

The spillway is 71 feet long, located 90 feet from the left abutment and has a crest elevation of 311.0. It is a broad-crested masonry weir of rectangular cross-section with a masonry approach channel and a downstream face of stepped masonry. Extending from the downstream side of the masonry face are stepped masonry wingwalls at each end of the spillway. At the base of the spillway there is a cobble apron.

A brick gatehouse is located upstream and adjacent to the right end of the spillway and accessible by a steel framed footbridge. Two manually operated gate valves are operated from within the gate house. One valve regulates a 20 inch blow-off, which presently acts as a low-level outlet, and the other regulates a 12 inch supply main which once led to the Bradley Hubbard Company, but now is terminated.

c. Size Classification - (SMALL) - The dam impounds 216 acre-feet of water with the reservoir level to the top of the dam, which at elevation 312.0, is 16.5 feet above the streambed of Harbor Brook. According to recommended guidelines, a dam with this height and maximum storage capacity is classified as small in size.

d. Hazard Classification - (HIGH) - If the dam were breached there is potential for loss of more than a few lives and extensive property damage to the George Hunter Golf Course and at least two homes on Westfield Road 3,500 feet downstream from the dam. The golf course is expected to be inundated by 6.6 to 11.0 feet of water in the vicinity of the streambed. At the second impact area, one house located 7.6 feet above the stream would be inundated by 3.4 feet of water and another house located 8.8 feet above the stream would experience up to 2.2 feet of water in the first floor. In addition, it is expected that Westfield Road would experience some flooding.

e. Ownership - City of Meriden
Department of Public Works
City Hall
Meriden, CT 06450
Bruce Marks (Director) (203)-634-0003

f. Operator - Owner (See Ownership, above)

g. Purpose - Originally for water supply, presently used for recreation.

h. Design and Construction History - The following information is believed to be accurate, based on the available data and correspondence and an interview with the owner of the dam. The dam was constructed about 1891 by James Kane and Sons, Builders, to supply water to the downstream factories. The dam was raised 3.5 feet and the 115 foot earth embankment and concrete corewalls were added about 1912. This work was performed by Leonardo Suzio, Contractor. There is no record of repairs or other alterations other than the raising in 1912.

i. Normal Operational Procedures - There are no formal operational procedures followed at the dam. The 20 inch low-level outlet is kept partially open. The 12 inch supply line has been terminated and is not functional.

1.3 PERTINENT DATA

a. Drainage Area - The drainage area is 0.59 square miles of mostly wooded, rolling to mountainous terrain located in the Quinnipiac River Basin. Approximately 8,500 feet upstream from the reservoir, there is a 700 foot long ungated conduit which diverts water into the Bradley Hubbard Reservoir and significantly enlarges the drainage area.

b. Discharge at Damsite - Discharge is over the spillway and through the 20 inch low-level outlet.

1. Outlet Works: 20 inch low-level outlet invert el. Not known	40 cfs (pond level at top of dam)
12 inch supply main:	N/A
2. Maximum flood at damsite:	Unknown
3. Ungated spillway capacity @ top of dam el. 312.0:	223 cfs
4. Ungated spillway capacity @ test flood el. 312.9:	604 cfs
5. Gated spillway capacity @ normal pool:	N/A
6. Gated spillway capacity @ test flood:	N/A
7. Total spillway capacity @ test flood el. 312.9:	604 cfs
8. Total project discharge @ test flood el. 312.9:	1325 cfs

c. Elevations - (NGVD based on assumed spillway elevation, See Sheet B-1).

1. Streambed at toe of dam:	295.5± ft.
2. Bottom of cutoff:	N/A
3. Maximum tailwater:	N/A
4. Normal pool:	311.0 ft.
5. Full flood control pool:	N/A
6. Spillway crest (ungated):	311.0 ft.

7. Design surcharge (original design):	Not known
8. Top of dam:	312.0 ft. (masonry) 313.0 ft. (embankment)
9. Test flood surcharge:	312.9 ft.

d. Reservoir Length (feet)

1. Normal pool:	3340 ft.
2. Flood control pool:	N/A
3. Spillway crest pool:	3340 ft.
4. Top of dam pool:	3400 ft.
5. Test flood pool:	3440 ft.

e. Reservoir Storage (acre-feet)

1. Normal pool:	180 acre-ft.
2. Flood control pool:	N/A
3. Spillway crest pool:	180 acre-ft.
4. Top of dam pool:	216 acre-ft.
5. Test flood pool:	230 acre-ft.

f. Reservoir Surface (acres)

1. Normal pool:	35 acres
2. Flood control pool:	N/A
3. Spillway crest pool:	35 acres
4. Top of dam pool:	35.8 acres
5. Test flood pool:	36 acres.

g. Dam

1. Type:	masonry core section with earth embankment slopes
2. Length:	340 ft.
3. Height:	16.5 ft.
4. Top width:	7.0 ft.

5. Side slopes:	1.5H to 1V (upstream) 2.0H to 1V (Downstream)
6. Zoning:	N/A
7. Impervious core:	Masonry core possibly to bedrock
8. Cutoff:	N/A
9. Grout curtain:	N/A
10. Other:	115 foot long earth embankment at right end. A 175 foot long concrete corewall at right end of masonry and 30 foot long concrete corewall at left end of masonry

h. Diversion and Regulating Tunnel N/A

i. Spillway

1. Type:	Broad-crested stone masonry rectangular weir
2. Length of weir:	71 ft.
3. Crest elevation:	311.0 ft.
4. Gates:	N/A
5. Upstream channel:	1.5H to 1V gravel
6. Downstream channel:	original streambed
7. General:	N/A

j. Regulating Outlets - The outlet is a 20 inch low-level outlet (blow-off). An abandoned 12 inch supply main still extends through the masonry and earth fill section.

1. Invert:	Low-level outlet	Unknown
	Supply main	N/A
2. Size:	Low-level outlet	20 inch
	Supply main	12 inch
3. Description:		Cast iron pipes

4. Control mechanism:	Manually operated handwheel pedestal, gate valve
5. Other:	Supply main abandoned. Actual length of pipe or where it terminates is unknown.

SECTION 2: ENGINEERING DATA

2.1 DESIGN

Available data consists of a plan accompanied by a contract and specifications between International Silver Co. in partnership with the Bradley and Hubbard Mfg. Co. and Leonardo Suzio, Contractor in reference to the raising of the dam; correspondence concerning an inspection of the dam on June 10, 1965 by John J. Mozzochi and Associates of Glastonbury Ct; and correspondence concerning an inspection of the dam on April 12, 1973 by Buck and Buck Engineers of Hartford, Connecticut. All correspondence is available from the State of Connecticut Department of Environmental Protection. The specifications and plan are available at the Town Hall, Meriden, Connecticut.

The drawings and correspondence indicate the design features stated previously in this report. There were no engineering values, assumptions, test results or calculations available for the original dam design or the 1912 raising of the dam.

2.2 CONSTRUCTION

There is no data available for the original construction of the dam or subsequent raising of the dam in 1912.

2.3 OPERATIONS

No operation records are known to exist.

2.4 EVALUATION

a. Existing Data - Existing data was provided by the State of Connecticut Department of Environmental Protection and the owner. The owner also made the project available for visual inspection.

b. Adequacy - The limited amount of detailed engineering data available was generally inadequate to perform an in-depth assessment of the dam, therefore, the final assessment of this dam must be based primarily on visual inspection, performance history, hydraulic computations of spillway capacity and approximate hydrologic judgements.

c. Validity - A comparison of record data and visual observations reveals no observable significant discrepancies in the record data.

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General - The condition of the project is fair based upon our visual inspection on May 12, 1980. The inspection revealed several areas requiring maintenance and monitoring. At the time of the inspection, the pond level was at elevation 311.0, i.e. 1.0 ft. below the top of the dam with a small amount of water flowing over the masonry spillway.

b. Dam

Top of Dam - The masonry coping contains cracks and deterioration within the mortar joints especially to the left side of the spillway. There is also vegetation growing from some of these cracks. The top of the earth embankment portion of the dam is bare and shows evidence of erosion (photos 1 and 2).

Upstream Slope - The upstream earthfill of the original dam section was below the water surface level therefore it could not be evaluated. The upstream slope of the embankment portion of the dam is irregular and badly eroded (Photo 2). Riprap had been removed or displaced from the embankment.

Downstream Slope - To the left of the spillway the slope is overgrown with large trees, brush and tall grass including numerous animal burrows (Photo 5). At the toe of the slope there is a seep of 5 gpm and a large wet area. The water from this seep was clear and flows toward the spillway channel. To the right of the spillway the slope is primarily covered by tall grass although some trees, tree stumps and brush exist near the spillway and channel area (photo 1). Animal burrows are evident in this area also. Extensive erosion has occurred behind the right masonry wingwall forming a large gully several feet deep. There is a large wet area at the toe to the right of the spillway from which a small stream develops, flowing at a rate of 4-6 gpm toward the spillway channel.

Spillway - The masonry spillway crest is in fair condition although there are some cracks and seepage through the masonry joints (photo 5). The approach channel is clear and free of obstructions. The training walls adjacent to the spillway crest show signs of slight erosion. Grasses and vines are growing from many of the joints in the masonry. Mortar is also missing from many of the joints (Photo 5). Seepage was observed from the joints of both training walls with flows averaging less than 1 gpm. The downstream face of the spillway is in fair condition although the masonry is a little eroded (Photo 4). The discharge channel is filled with debris and overgrown with trees and brush (Photos 4, 5 and 6).

c. Appurtenant Structure - The exterior of the brick gatehouse is in fair condition. In several areas, the concrete at the base of the brickwork, is deteriorated and the steel sheeting covering the vertical sides of the concrete base is pulled away or missing from the concrete. The wood decking, of the steel framed foot bridge is missing which makes entry to the gatehouse difficult. The wood floor inside the gatehouse is badly deteriorated. The handwheel of one of the valves has been removed from the pedestal.

d. Reservoir Area - The area surrounding the pond is generally wooded and undeveloped. There are steep wooded hills to the east and northwest and a golf course to the west of the dam.

e. Downstream Channel - The downstream channel is the natural streambed of Harbor Brook. The channel was very overgrown with large trees, brush, uprooted trees, and assorted grasses. It is difficult to define the actual channel.

3.2 EVALUATION

Based upon the visual inspection, the project is assessed as being in fair condition. The following features which could influence the future condition and/or stability of the project were identified.

1. Significant seepage through the masonry has and will continue leaching the cement mortar joints thus weakening the masonry and decreasing stability. Freezing and thawing of this seepage could result in displacement of the stonework and/or possible failure of the masonry.

2. Vegetation growing through the masonry joints could lead to displacement and/or possible failure of the masonry.

3. Cracks between the newer concrete and the upstream face of the original masonry (See Sheet B-1), allow water to flow through the masonry section thus possibly leading to adverse seepage through the dam.

4. The lack of riprap or other suitable protective cover on the top and upstream slope of the embankment portion of the dam will permit further erosion which may possibly result in failure of the structure.

5. Trees, brush and burrowing animals could promote piping and/or seepage by creating flow paths, either along root systems or through holes, in the embankment. Trees, if uprooted may produce depressions which may be critical to the stability of the dam.

6. Seepage and wet areas at the toe of the downstream embankment could increase and lead to instability if not properly monitored.

7. The wood decking is missing from the footbridge leading to the gatehouse, making it difficult as well as dangerous to get into the gatehouse.

SECTION 4: OPERATIONAL AND MAINTENANCE PROCEDURES

4.1 OPERATIONAL PROCEDURES

a. General - No formal program of operation is in effect. It was reported that the low-level outlet was opened in the summer of 1979 to provide water to a public swimming area downstream.

b. Description of any Warning System in Effect - No formal warning system is in effect.

4.2 MAINTENANCE PROCEDURES

a. General - There is no formal program of maintenance or inspection at the dam.

b. Operating Facilities - No formal program for maintenance of operating facilities is in effect.

4.3 EVALUATION

Operation and maintenance procedures are not performed. A formal program of operation and maintenance procedures should be implemented, including documentation to provide complete records for future reference. Also, an emergency action plan as well as a formal downstream warning system should be developed and implemented within the time frame indicated in Section 7.1c. Remedial operation and maintenance recommendations are presented in Section 7.3.

SECTION 5: EVALUATION OF HYDRAULIC/HYDROLOGIC FEATURES

5.1 GENERAL

The Bradley Hubbard Reservoir Dam drainage area is 0.59 square miles of wooded rolling to mountainous terrain. An ungated conduit upstream, diverts water to the reservoir and substantially increases the drainage area (See Sheet D-1).

The dam is basically a low surcharge storage - high spillage type project. The available storage reduces the outflow from a Probable Maximum Flood (PMF) from 1500 cubic feet per second (cfs) to 1325 cfs and the ½ PMF outflow from 750 cfs to 620 cfs.

5.2 DESIGN DATA

No computations could be found for the original design of the dam or the subsequent raising.

5.3 EXPERIENCE DATA

The maximum discharge at this dam site is unknown and no information was found to indicate that there have been any problems (including overtopping) arising at the dam.

5.4 TEST FLOOD ANALYSIS

Based upon the U.S. Army Corps of Engineers "Preliminary Guidance for Estimating Maximum Probable Discharges" dated March, 1978; the watershed classification (rolling to mountainous) and the watershed area of 0.59 square miles, a PMF of 1500 cfs or 2550 cfs per square mile is estimated at the damsite. In accordance with the size (small) and hazard (high) classification, the range of test floods to be considered is from the ½ PMF to the PMF. Based on the hazard potential associated with a breach of the dam, the test flood for Bradley Hubbard Reservoir Dam is selected as equivalent to the PMF. The pond level at the start of the test flood is considered to be at elevation 311.0, which is at the spillway crest. Peak inflow to the reservoir at the test flood is 1500 cfs; peak outflow is 1325 cfs with the dam overtopped by 0.9 feet. Based on hydraulics computations, the spillway capacity to the top of the dam is 223 cfs which is equivalent to 17% of the routed test flood outflow (Appendix D-6).

5.5 DAM FAILURE ANALYSIS

Utilizing the Corps of Engineers April 1978 "Rule of Thumb Guidance for Estimating Downstream Dam Failure Hydrographs", the peak failure outflow due to a breach of the dam is estimated to be 11,700 CFS with an estimated flood depth of 7 Ft. immediately downstream of the dam. The flood routing was performed for peak failure outflow with pool at top of dam. The prefailure flow in the brook is estimated to be 223 CFS and flood stages are estimated to increase by 4.3 Ft. and 7.9 Ft. at the initial and second impact areas respectively.

The estimated peak flow rates and peak flood depths at four sections downstream of the dam resulting from a dam failure are:

D/S Section (Ft. From Dam)	Flow (CFS)	Flood Depth (Ft)	Velocity (FPS)
At Dam	11,700	7	-
1350	10,100	9.6	11
1950	9,400	6.6	11
2450	8,500	4.5	6
3800	6,000	11	5

As discussed in Appendix D (D-23 & 24), a flood of this magnitude would inundate a significant portion of George Hunter Golf course and flood at least two houses on Westfield Road. The flood depth in the golf course, considered as initial impact area, would vary from 6.6 ft. to 11 ft. in the vicinity of the existing channel. At the second impact area in the vicinity of Westfield Road, the house located north of the road has its first floor 7.6± ft. above the channel bed, and would be inundated with 3.4± ft. of flood water. Similarly, the house located south of the Westfield Road would be ununda ted with 2.2± ft. of water, since its first floor elevation is 8.8± ft. above the channel bed. In addition, it is expected that three culverts would be damaged and Westfield Road would be inundated with 2.5± ft. of water at two locations.

Based upon the hydraulic and hydrologic analysis, the dam has a high hazard classification with a potential for loss of more than a few lives upon failure of the dam.

SECTION 6: EVALUATION OF STRUCTURAL STABILITY

6.1 VISUAL OBSERVATIONS

The dam is basically in two sections. The main section is the original stone masonry dam with earth fill added on the downstream side, and earth and rock fill added on the upstream side. The second section is the newer part of the dam added in 1912. This is an earth embankment with a concrete corewall, both of which abut the right end of the masonry core. This section was added to fill a low area resulting from raising the original dam 3.5 feet in 1912. The concrete corewall at the right end of the dam extends for 175 feet, through the earth embankment section and into the natural earth abutment. Another 30 foot section of concrete corewall was also added to the left end of the dam. The dam was raised by removing the cap stones, placing 3.5 feet of concrete on the top and replacing the cap stones (See Sheet B-1, Section B-B). The inclination of the rock fill on the upstream slope is 1.5 horizontal to 1 vertical and the inclination of the downstream slope is 2.0 horizontal to 1 vertical.

The visual inspection revealed a series of maintenance and repair related problems which, if not corrected, could compromise the stability of the dam. In summary, these would include: 1) cracking of the masonry joints and between the newer concrete and the original masonry, allowing seepage to occur through the masonry cap stones and through the spillway section, 2) seepage of approximately 5 gpm (clear water flowing) and a large wet area at the right and left ends of the toe of the dam, 3) animal burrows, erosion and fairly large trees on the downstream slopes, 4) erosion and lack of slope protection on the earth embankment section to the right end of the dam, 5) the poor condition of the gatehouse and operating facilities. See Section 7 for recommendations and remedial measures.

6.2 DESIGN AND CONSTRUCTION DATA

The drawings and data available and listed in Appendix B were not sufficient to perform an in depth stability analysis of the dam. No engineering assumptions, data or calculations could be found for the original design of the dam.

6.3 POST CONSTRUCTION CHANGES

Post construction changes of the project consisted of raising the crest of the dam 3.5 feet and the addition of 115 feet of embankment and a concrete corewall at each end of the dam to increase storage.

6.4 SEISMIC STABILITY

The project is in Seismic Zone 1 and according to the Recommended Guidelines, need not be evaluated for seismic stability.

SECTION 7: ASSESSMENT RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Condition - Based upon the visual inspection of the site and past performance, the project appears to be in fair condition. However, there are areas which require maintenance, repair and monitoring.

Based upon the Army Corps of Engineers' "Preliminary Guidance for Estimating Maximum Probable Discharges" dated March, 1978, the watershed classification and hydraulic/hydrologic computations, peak inflow to the lake at the test flood is 1500 cubic feet per second (cfs); peak outflow is 1325 cfs with the dam overtopped 0.9 feet. Based upon our hydraulic computations, the spillway capacity to the top of dam is 223 cfs, which is equivalent to approximately 17% of the routed test flood outflow.

b. Adequacy of Information - The information available is such that an assessment of the condition and stability of the project must be based solely on visual inspection, past performance and sound engineerng judgement.

c. Urgency - It is recommended that the measures presented in Section 7.2 and 7.3 be implemented within one year of the owner's receipt of this report.

7.2 RECOMMENDATIONS

It is recommended that further studies be made by a registered professional engineer qualified in dam design and inspection pertaining to the following items. Recommendations made by the engineer should be implemented by the owner.

1. A detailed inspection of the spillway and spillway channel when no water is flowing over the spillway to check for seepage through the masonry and erosion of the cobble apron at the base of the spillway.

2. Determination of the origin and significance of seepage and wet areas at the toe of the downstream embankment.

3. Removal of all trees, tree stumps, and brush from the embankments and the spillway channel. This should include removal of root systems, proper backfilling and regrading of eroded areas.

4. The upstream slope of the embankment portion of the dam should be regraded, riprap placed on the upstream slope and slope protection placed on the top of the embankment which will resist the frequent foot traffic.

5. A hydraulic/hydrologic analysis should be performed to more accurately determine the adequacy of the existing project discharge and the overtopping potential.

6. Sealing the cracks between the newer concrete section and the original masonry to prevent seepage through this area.

7.3 REMEDIAL MEASURES

a. Operation and Maintenance Procedures - The following measures should be undertaken by the owner within the length of time indicated in Section 7.1.c, and continued on a regular basis:

1. Round-the-clock surveillance should be provided during periods of heavy precipitation or high project discharge. A formal emergency preparedness plan should be devised so in the event of an emergency, evacuation may be implemented in a prompt and organized manner.

2. A formal program of operation and maintenance procedures should be instituted and fully documented to provide accurate records for future reference. The maintenance procedures should include a monthly inspection by the owner or owner representative.

3. A comprehensive program of inspection by a registered professional engineer qualified in dam inspection should be instituted on a biennial basis.

4. The vegetation should be removed from the masonry joints and all masonry repointed.

5. The gully on the downstream slope along the right spillway training wall, and any other visible slope erosion, should be backfilled with suitable material and proper slope protection placed.

6. Decking should be replaced on the footbridge to the gatehouse and fencing to protect against vandalism installed.

7. Flooring should be replaced in the gate house.

8. The gate house door should be repaired.

9. The gate valve mechanisms should be repaired, cleaned, lubricated, and painted.

10. The discharge channel should be cleared of trees, brush and logs, and the cobble apron repaired to prevent erosion at the base of the spillway during high spillway discharge.

11. Animal burrows should be evacuated, properly backfilled and slope protection placed.

7.4 ALTERNATIVES

This study has identified no practical alternatives to the above recommendations.

APPENDIX A

INSPECTION CHECKLIST

VISUAL INSPECTION CHECK LIST
PARTY ORGANIZATION

PROJECT Bradley Hubbard Reservoir Dam DATE: May 12, 1980

TIME: 9:30 - 10:30 Am

WEATHER: Cloudy 55°

W.S. ELEV. 311 U.S. _____ DN.S

PARTY:	INITIALS:	DISCIPLINE:
1. Peter Heynen	PH	Cahn, Geotechnical
2. Miron Petrovsky	MP	Cahn, Geotechnical
3. Jay Costello	JC	Cahn, Geotechnical
4. Jeffery Borne	JB	Cahn, Geotechnical
5. Dr. Murali Atluru	MA	Diversified Tech, Hydraulics
6.		

PROJECT FEATURE	INSPECTED BY	REMARKS
1. Earth Embankment	PH, MP, JC, JB, MA	A-2
2. Spillway	PH, MP, JC, JB, MA	A-3
3. Gatehouse	PH, JC	A-4
4.		
5.		
6.		
7.		
8.		
9.		
10.		
11.		
12.		

PERIODIC INSPECTION CHECK LIST

PROJECT Bradley Hubbard Reservoir Dam DATE 5-12-80

PROJECT FEATURE Earth Embankment BY EH, MP, JC, JB, MA

AREA EVALUATED		CONDITION
DAM EMBANKMENT		
Crest Elevation		312.0 (masonry) 313.0 (earth)
Current Pool Elevation		311.0
Maximum Impoundment to Date		Not Known
Surface Cracks		Joint crack in masonry
Pavement Condition		N/A
Movement or Settlement of Crest		None Observed
Lateral Movement		
Vertical Alignment		Appears good
Horizontal Alignment		
Condition at Abutment and at Concrete Structures		Fair
Indications of Movement of Structural Items on Slopes		None observed
Trespassing on Slopes		Excessive trespassing on top of embankment section
Sloughing or Erosion of Slopes or Abutments		U/s slope embankment section
Rock Slope Protection-Riprap Failures		U/s slope embankment section - no riprap
Unusual Movement or Cracking at or Near Toes		None observed
Unusual Embankment or Downstream Seepage		wet areas at D/s toe of dam to each side of spillway.
Piping or Boils		None observed
Foundation Drainage Features		N/A
Toe Drains		N/A
Instrumentation System		N/A

PERIODIC INSPECTION CHECK LIST

PROJECT Bradley Hubbard Reservoir Dam DATE 5-12-80

PROJECT FEATURE Spillway BY PH, MP, JG, JB, MA

AREA EVALUATED		CONDITION
OUTLET WORKS-SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS		
a) Approach Channel		
General Condition		Good
Loose Rock Overhanging Channel		No
Trees Overhanging Channel		No
Floor of Approach Channel		Stones on bottom - clear
b) Weir and Training Walls		
General Condition of Concrete		Fair
Rust or Staining		No
Spalling		little eroded, mortar leached from joints.
Any Visible Reinforcing		No
Any Seepage or Efflorescence		Seeps through joints (<1gpm) possible undermining at toe
Drain Holes		
c) Discharge Channel		
General Condition		Poor
Loose Rock Overhanging Channel		No
Trees Overhanging Channel		Area heavily wooded and overgrown.
Floor of Channel		Heavily wooded, brush, logs,
Other Obstructions		

PERIODIC INSPECTION CHECK LIST

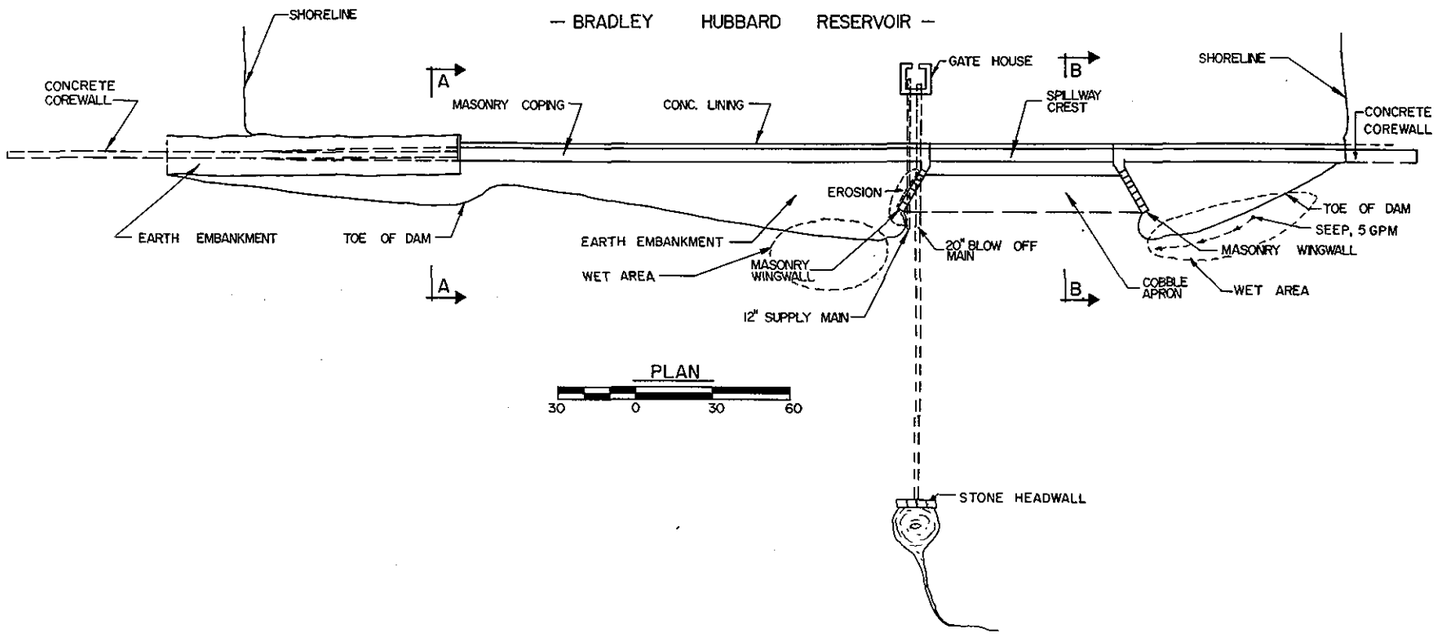
PROJECT Bradley Hubbard Reservoir Dam DATE 5-12-80

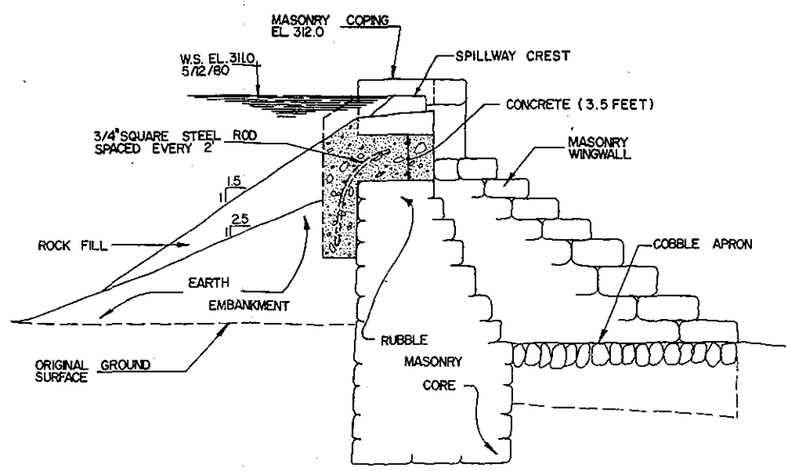
PROJECT FEATURE Gatehouse BY PH, MP, JC, JB, NA

AREA EVALUATED		CONDITION
OUTLET WORKS-CONTROL TOWER		
a) Concrete and Structural		
General Condition		Poor
Condition of Joints		Cracking of concrete foundation and brick superstructure
Spalling		Some
Visible Reinforcing		None observed
Rusting or Staining of Concrete		
Any Seepage or Efflorescence		
Joint Alignment		N/A
Unusual Seepage or Leaks in Gate Chamber		None observed
Cracks		yes - brickwork and concrete foundation
Rusting or Corrosion of Steel		yes - steel sheeting around foundation
b) Mechanical and Electrical		
Air Vents		N/A
Float Wells		
Crane Hoist		
Elevator		
Hydraulic System		
Service Gates		
Emergency Gates		
Lightning Protection System		
Emergency Power System		
Wiring and Lighting System		

APPENDIX B

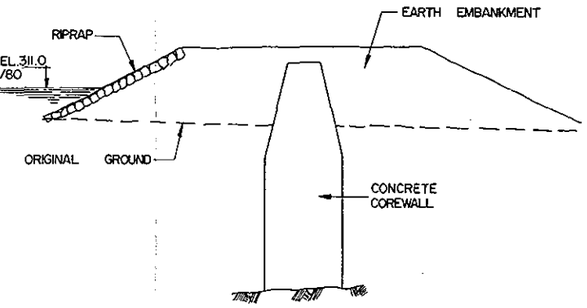
ENGINEERING DATA AND CORRESPONDENCE





SECTION B-B

5 0 5 10



SECTION A-A

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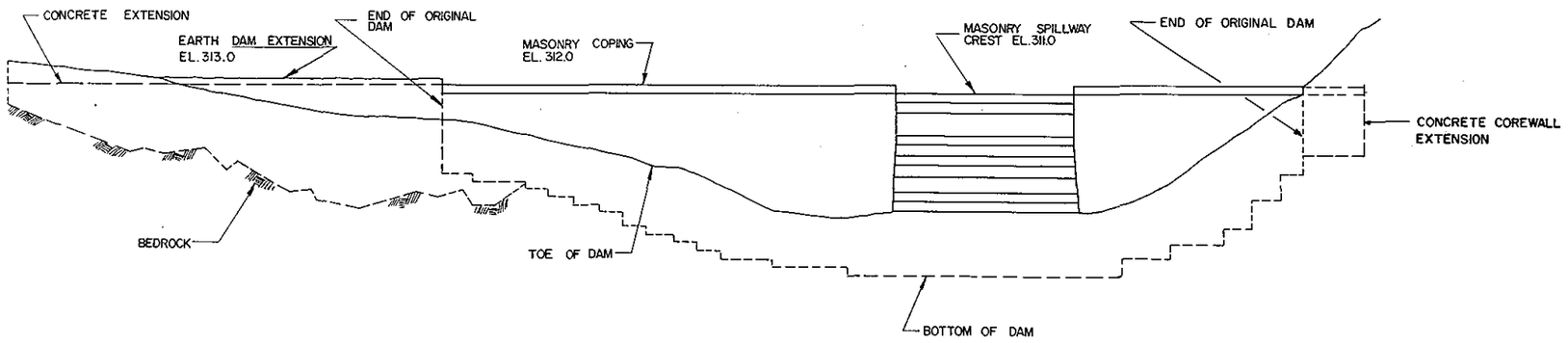


ELEVATION

30 0 30 60 HORIZONTAL

10 0 10 20 VERTICAL

NOTES:

1. THIS DRAWING TRACED FROM A PLAN TITLED "DAM AT STORAGE RESERVOIR" PREPARED FOR THE BRADLEY & HUBBARD MFG. CO. AND THE INTERNATIONAL SILVER CO. NO DATE OR INDICATION WHO PREPARED THE PLAN WAS AVAILABLE.

2. ALL ELEVATIONS ARE N.G.V.D. BASED ON AN ASSUMED SPILLWAY CREST ELEVATION. THE WATER SURFACE ELEVATION OF 311.0 SHOWN ON THE 1972 MERIDEN U.S.G.S. QUADRANGLE MAP WAS ASSUMED TO BE THE ELEVATION OF THE SPILLWAY CREST. ALL OTHER ELEVATIONS ARE REFERENCED TO THE SPILLWAY CREST ELEVATION.

CAHN ENGINEERS INC. WALLINGFORD, CONNECTICUT ENGINEER	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

PLAN, ELEVATION AND SECTION

BRADLEY HUBBARD RESERVOIR DAM

HARBOR BROOK MERIDEN, CONNECTICUT

DRAWN BY	CHECKED BY	APPROVED BY	SCALE: AS NOTED	
M.D.S.man	JAC	AHH	DATE: SEPT. 1980	SHEET B-1

BRADLEY HUBBARD RESERVOIR

EXISTING PLANS

"International Silver Company and Bradley and Hubbard Mfg. Co., Dam at Storage Reservoir"

No date or signatures.

SUMMARY OF DATA AND CORRESPONDENCE

DATE	TO	FROM	SUBJECT	PAGE
Sept. 25 1912			Contract and specification between owners, International Silver Co. and The Bradley and Hubbard Co., and contractor, Leonardo Suzio, in reference to raising the dam	B-3
June 11, 1963	File	State Board for the Supervision of Dams	Inventory Data	B-12
May 3, 1965	John J. Mozzochi and Associates	William P. Sanders Water Resources Commission	Inspection Request	B-13
June 14, 1965	William P. Sanders Water Resources Commission	John J. Mozzochi and Associates	Results of Dam Inspection	B-14
June 21, 1965	BeElden A. Philbrook Soil Conservation	William P. Sanders	Cover letter for dam inspection by John J. Mozzochi and Associates	B-15
April 26, 1973	Victor Galgowski Conn. Dept of Environmental Protection	Buck and Buck Engineers James A. Thompson	Visual dam inspection results	B-16
May 10, 1973	Mayor Abraham G. Grossman - Meriden, Ct.	Victor Galgowski	Inspection report and recommendation.	B-17
May 15, 1973	Dan W. Lufkin Conn. Dept. of Environmental Protection	Mayor Abraham G. Grossman	Request of detailed recommendations	B-18
July 8, 1974	Douglas Mcostle Conn. Dept. of Environmental Protection	Mayor John D. Quine - Meriden, Ct.	Request of detailed recommendations	B-19
July 17, 1974	Mayor John D. Quine	Victor Galgowski	Detailed recommendations	B-21

AND

THE BRADLEY & HUBBARD MFG. CO.

R A I S I N G P R E S E N T D A M

MERIDEN, CONN.

THIS AGREEMENT made and concluded this 25th day of September 1912, by and between The International Silver Co., and The Bradley & Hubbard Mfg. Co., corporations organized under the Laws of the State of Connecticut, and having their place of business in the City of Meriden in said State, Party of the First Part, and Leonardo Suzio Contractor, Party of the Second Part,

WITNESSETH:

(A) That the said party of the second part has agreed and by these presents does agree with the said party of the first part, for the consideration hereinafter mentioned and contained, to furnish all necessary labor and tools, and to construct in a substantial and workmanlike manner, the additions to the present dam of the party of the first part in the Town of Meriden, in the manner, and under the conditions hereinafter specified; and has further agreed that the said party of the first part shall be, and is, hereby authorized to appoint its engineer, and such other person or persons as it may deem proper, to inspect materials furnished and the work done, and to see that the same correspond with the specifications hereinafter set forth, to wit:

(B) S P E C I F I C A T I O N S

B-3

(1) WORK TO BE DONE.

The work contemplated and provided for in this

house about three feet and five inches; the construction of additional embankments on both sides of the present dam; and the construction of an earth dam with concrete core wall at each end of the present dam.

All materdals shall be provided by the contractor, excepting that stone suitable for use in the work may be obtained on the property of the International Silver Co., and The Bradley & Hubbard Mfg. Co.

All work shall be done strictly in accordance with the plan on file in the office of The Bradley & Hubbard Mfg. Co.

(2). ADDITION TO THE PRESENT DAM.

The coping stones on the present dam and the two upper courses of stone on the spillway shall be removed and placed at convenient places on the embankment. All loose stone and mortar on top of the masonry wall shall be removed so as to give a good bond between the new concrete and masonry.

A concrete wall shall be built on top of this masonry of the size and dimensions shown on the plan. The concrete facing on the back of the dam shall be carried down into the water from one to two feet as directed by the engineer. A 3/4" square steel rod 8 feet long bent as shown on the plans shall be placed in the concrete and spaced every two feet as shown on the plan.

The dam shall be divided in sections of about fifty feet or such length that all the concrete in the section can be placed in the same day. At the end of each section a vertical groove shaped as directed by the engineer, shall be made in the concrete to form an expansion joint. The end of each section shall be oiled, before the concrete in the next section is placed.

planed planks. Adjoining planks of the same mold shall be of the same thickness with the edges beveled, or tongues and grooved to make the joints water tight.

All molds shall be thoroughly cleaned of all cement before being used. Deformed, broken, or defective molds shall be repaired or removed from the work.

The molds shall be allowed to remain in place a sufficient length of time to allow the concrete to set; and they shall be constructed in such a manner that they can be readily removed without jarring or cracking the concrete.

Forms may be omitted for the vertical sections of the core walls provided that the excavation is made truly to the widths shown on the plan.

(11) CEMENT.

All cement used in the work shall be of the best quality of Portland cement of a brand that has an established reputation for uniformity and quality. It shall be dry and free from lumps; be ground so finely that ninety-two per cent will pass a sieve with ten thousand meshes to the square inch; and parts of neat cement one half inch in thickness and three inches in diameter with thin edges shall not crack in setting or when immersed in water maintained at a temperature of one hundred and seventy-five degrees Fahrenheit. The color shall remain uniform over the whole surface of the cement after becoming hard, and not show yellowish spots, whether the pats are set in air or in water.

Briquetts, molded of neat cement, shall have a tensil strength of at least one hundred and fifty pounds per square inch after twenty four hours immersion in water - the briquett to be placed in water immediately after being set - and at least five hundred pounds after one day in air and six days in water; and shall show a gradually increasing strength after that time.

the wall shall be covered with a thick bed of mortar and the coping stones set thereon. If so directed by the engineer, the paving in the cobble apron next to the spillway shall be removed for a width of about three feet, and a section of concrete about two feet deep be thoroughly rammed into the space.

(3) RAISING THE GATE HOUSE.

The present gate house and bridge shall be raised approximately three feet five inches as shown on the plan. The gate house shall be jacked up with timbers, and the foundations carried up with stone to the under side of the sand stone water table. The ladder in the gate well and the gate stems shall be lengthened to fit the new floor level.

(4) EXCAVATION.

The earth on the site of the core walls shall be excavated down to rock, and a trench shall be excavated in the rock, if necessary, to a sufficient depth to secure a good foundation for the concrete. The width of the excavation shall be the same width as the core wall.

On the site of the embankments for the core wall, all loam, stumps, roots, and other vegetable matter shall be grubbed out and removed from the entire area to be covered by the new work. The loam shall be piled at one side of the excavation and be used for surfacing the embankments.

The loam on the embankments in front of the present dam shall be removed before building the additional embankments.

The excavation for the concrete on back of the present dam shall be carried down into the selected material, and from one to two feet into the water, as directed by the engineer.

Wherever rock is encountered in the excavations it shall be stripped of earth and the engineer notified that he may cross-section the same.

Special care shall be taken in preparing the foundations for the core wall to shatter the surrounding rock as little as possible. All loose rock must be removed, ~~practicable without~~ without blasting.

Only ledge rock and boulders measuring more than one half of a cubic yard in volume shall be measured as rock.

(6) EMBANKMENTS.

After the loam and other soft material has been removed from the site of the embankments the earth beneath shall be loosened by plowing or harrowing to secure a bond between the natural soil and the new material.

Only selected material which will "pack" when moistened and tamped shall be used in the embankment on the water side of the core wall. At the back of the present dam the rip rap shall be removed as low as the water in the reservoir will permit, and be replaced with selected material.

The down stream side of the core wall embankment, and the embankment in front of the present dam may be made of gravel or other material which will form a solid bank. No stumps or other vegetable substances, and no stones which are too large to be thoroughly bedded by the tampers shall be used.

The material for the embankments shall be taken from the excavation for the core walls or from the reservoir basin below the flow line. It shall be deposited in horizontal layers not exceeding six inches in depth, be sprinkled with water, and thoroughly tamped with heavy iron tampers. The amount of water used, and the extent to which the material shall be tamped shall be regulated by the engineer

slope of one on two. The up stream or water face of the embankment shall be covered with rip rap to a level two feet above the level of the top of the spillway. The remainder of the embankments shall be covered with a layer of black loam to a depth of at least six inches and be well seeded with grass of the variety determined by the engineer.

The rip rap shall be composed of sound and durable stones and be of such size and shape as to form a facing at least one foot in depth. The stones shall be set by hand close together, the interstices between the larger stones being chinked up with spalls and small stones to make a smooth and compact surface. After the rip rap is laid sand or gravel shall be spread over the surface and be broomed into the joints until all spaces between the stones are solidly filled.

(7) ROCK FILL.

The rip rap on the upstream slope of the present dam shall be covered to a depth of about two feet with broken rock of a size which can be handled by one man. The face of this rock fill shall be graded on a slope of one on one and one half.

(8) CORE WALL.

A concrete core wall shall be built at both ends of the present dam, and be extended into the natural bank at each side as directed by the engineer. It shall be founded on solid rock, and be carried up within one foot of the top of the embankment.

(9) CONCRETE.

All concrete used in the work shall be composed of one part of cement, two and one half parts of fine, and four and one half parts of coarse aggregate.

the surface shall be washed with a thin grout of cement and sand and be floated with soft wooden floats until the surface is smmoth and hard.

The concrete and mortar shall be made in concrete mixing machines of approved form. The ingredients for a "batch" shall be assembled in suitable measuring boxes before being placed in the mixer. The cement and sand shall first be mixed to a thin mortar, the stone afterwards added, and the mixing continued until a homogeneous mixture is obtained.

The concrete shall be mixed "wet", but the exact amount of water shall be determined by the engineer. It shall be deposited in place immediately after being mixed, and be thoroughly compacted by tamping, and by spading along the sides of the forms.

No work in concrete shall be done when the temperature is below freezing.

The surface of the rock, the top of the present dam, and concrete which has set shall be covered with a thin layer of mortar before the placing of any concrete thereon. The mortar shall be composed of one part of cement and two and one half parts of sand.

Where new masonry is joined to old, the surfaces of the old concrete shall be cleaned of all laitance, and soft or loose cement, by scrubbing with wire brushes, and be thoroughly washed.

(10) FORMS.

The forms or molds for the different parts of the work shall be built of the exact shape of the structures which they are to form; and be of sufficient strength and rigidity to permit of the concrete being thoroughly tamped and compacted without springing or warping them from that shape.

cient amount of cement on hand to permit of its being tested before being used.

The cement shall be kept stored in a tight shed so constructed that the cement will be protected from the weather and from dampness from the ground.

A barrel of cement shall be reckoned as three hundred and eighty pounds net weight.

(12) AGGREGATES.

The fine aggregate shall be clean, sharp pit sand, free from clay or loam, or fine stone dust such as will pass a sieve with one quarter of an inch mesh.

The coarse aggregate shall be broken stone such as will pass a screen with two inch round holes and be rejected by a sieve with one quarter of an inch mesh. Only hard, durable stone will be accepted.

(13) BRICK.

The ~~brick~~ stone used in the gate house foundation shall be regular and uniform in shape and size, with full sharp corners, and be hard burned entirely through. They shall be thoroughly wet with water before being laid, and have full cement joints, at bed, sides, and ends, which shall be made at one operation and not by working the mortar in after the brick is laid. The joints shall be properly struck on the face of the work.

(14) STEEL.

The steel rods used in the work shall be of the size shown on the plan. They shall be placed in the work in such a manner as to be thoroughly covered with concrete; and shall be truly bent to the form directed by the engineer.

(15) PROTECTION OF WATERSHED.

All buildings for housing the men or animals employed on the work shall be built on land entirely off the watershed of the present reservoir. They shall be kept at

AND

THE BRADLEY & HUBBARD MFG. CO.

RAISING PRESENT DAM

MERIDEN, CONN.

ESTIMATE OF QUANTITIES.

Earth Excavation - - - - - - - - - - -	500 cu. yds.	~~75.00~~
Rock Excavation - - - - - - - - - - - -	10 cu. yds.	35.00
Rolled Earth Embankment - - - - - - -	1200 cu. yds.	~~1320.00~~
Concrete - - - - - - - - - - - - - - -	700 cu. yds.	4935.00
Rock Filling - - - - - - - - - - - - -	250 cu. yds.	~~187.50~~
Rip Rap - - - - - - - - - - - - - - - -	75 sq. yds.	~~75.00~~
Steel - - - - - - - - - - - - - - - - -	3000 lbs.	~~180.00~~
Coping stones to be moved - - - - - -	425 lin. ft.	~~403.75~~
Brick for gate house foundation - - -	7000	550.00
		~~7957.00~~

The above quantities are to be considered only as approximate. The International Silver Co., and The Bradley & Hubbard Mfg. Co. reserve the right of increasing or diminishing the same as may be deemed necessary by the engineer.

E

CT-132 1

No. ME 3

WATER RESOURCES COMMISSION
SUPERVISION OF DAMS
INVENTORY DATA

LONG 72-45.7
LAT 41-33.5

Inventoried By WPS

Date 11 JUNE 1963

Name of Dam or Pond BRADLEY HUBBARD RESERVOIR

Code No. QU 23.4 HR 5.6

Nearest Street Location WESTFIELD ROAD

Town MERIDEN

U.S.G.S. Quad. MERIDEN

Name of Stream HARBOR BROOK

Owner ~~CONNECTICUT LIGHT AND POWER CO~~ CITY OF MERIDEN

Address BERLIN
City Hall
Meriden
OL 7/73

Pond Used For WATER SUPPLY (?) DA 0.595SM

Dimensions of Pond: Width 500 FEET Length 3000 FEET Area ~~40~~ 34.0 ACRES

Total Length of Dam 280 FEET Length of Spillway 40 FEET

Location of Spillway EAST END OF DAM

Height of Pond Above Stream Bed 15 FEET

Height of Embankment Above Spillway 3=FEET 1'

Type of Spillway Construction MASONRY

Type of Dike Construction MASONRY

Downstream Conditions FIELDS, ROADS

Summary of File Data

Remarks

Would Failure Cause Damage? YES B=12 Class B

May 3, 1965

John J. Mozzochi and Associates
217 Hebron Avenue
Glastonbury, Connecticut

Gentlemen:

Under the terms of your contract as consultant to this Commission, will you please inspect and report on Bradley Hubbard Reservoir in Meriden. There is a proposed flood control project at Baldwin Pond immediately downstream and for this reason we would like to know the present condition of Bradley Hubbard Reservoir.

The Bradley Hubbard Reservoir is just east of Route 15 on the east side of the Meriden Quadrangle.

Very truly yours,

William P. Sander
Engineer - Geologist

WPS:jc

JOHN J. MOZZOCHI AND ASSOCIATES
CIVIL ENGINEERS

GLASTONBURY, CONN.
217 HEBRON AVENUE
PHONE 633-9401

PROVIDENCE 3, R. I.
196 DYER STREET
PHONE GASPEE 1-0420

JOHN J. MOZZOCHI
ASSOCIATES
OWEN J. WHITE
JOHN LUCHS, JR.
ECTOR L. GIOVANNINI

June 14, 1965

REPLY TO: Glastonbury

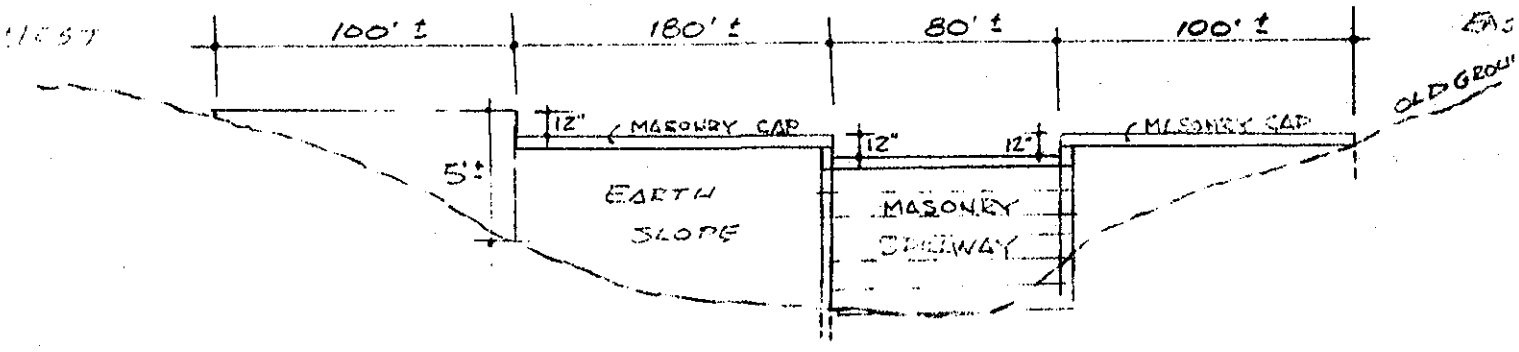
William P. Sander-Engineer - Geologist
Water Resources Commission
State Office Building
Hartford 15, Connecticut

Re: Our File 57-73-68
Bradley Hubbard Reservoir
Meriden, Connecticut

Dear Mr. Sander:

In accordance with your instructions of May 3, 1965, I made an inspection of the referenced dam on June 10th and found that it is in substantially good condition. There is some minor leakage noted around the spillway which appears to be due to the need of pointing of joints below the large sandstone capstones. This is a matter of routine maintenance only and should not be construed to be of any immediate importance.

The dam itself is about 460' overall consisting of an earthen dike about 100' long on the west end, a center masonry spillway section about 80' in length and 18' high with about a 12 inch freeboard, and two masonry capped abutment sections having concrete cores and earthen slopes, about 180' long on the west of the spillway and about 100' long on the east side, being apparently constructed to act as supplementary additional spillways, with about 12" freeboard to the earth dike on the west end.



In my opinion, this dam is in perfectly safe condition and should not be a cause for concern to any structure immediately downstream.

Very truly yours,

John J. Mozzochi

John J. Mozzochi and Associates
Civil Engineers

JJM:hk

June 21, 1965

Mr. BeElden A. Philbrook
U. S. Department of Agriculture
Soil Conservation Service
Old Bookstore Building
Route 195
Storrs, Connecticut

Dear Mr. Philbrook:

Enclosed is a copy of a report from one of our consulting engineers on the present condition of the Bradley Hubbard Reservoir in Meriden. Mr. John Curry of this office asked me to send you this copy in connection with the flood control project downstream.

We trust that this information will be of value to you.

Very truly yours,

William P. Sanders
Engineer - Geologist

WPS:js

enclosure

BUCK & BUCK

ENGINEERS

98 WADSWORTH STREET, HARTFORD, CONNECTICUT 06106

JAMES A. THOMPSON
ROBINSON W. BUCK
LAWRENCE F. BUCK

HENRY WOLCOTT BUCK
1931-1965
ROBINSON D. BUCK
1955-1956

COMM. 5713-76

April 26, 1973

Mr. Victor Galgowski,
Department of Environmental Protection,
Water Resources Division,
State Office Building,
Hartford, Connecticut 06106

Re: Bradley Hubbard Reservoir Dam
Meriden

Dear Vic:

We inspected the subject dam on April 12th, and found the cap stones in need of pointing. Leakage through the dam, under the cap stones is beginning to errode the downstream earthen face of the dam. We also noted woodchuck holes on the downstream slope. These holes should be plugged and the woodchucks eradicated.

The repointing of the cap stones should be done from the upstream side and it may have to include complete rebedding of some stones. All of this work may be considered ordinary maintenance that does not require a permit. I suggest that the owner notify your office when the work is being done so that you can make a follow-up inspection.

Sincerely,

BUCK & BUCK

James A. Thompson

JAT:fb

10 May 1973

The Honorable Abraham G. Grossman
City Hall
Meriden, Connecticut 06450

Re: Bradley Hubbard Reservoir Dam
Meriden

Dear Mayor Grossman:

A recent inspection, by one of our consultants, has indicated the need for some maintenance work on the subject dam.

In general, the cap stones need to be repointed from the upstream side and this may include complete rebedding of some stones. The present condition of the cap stones is allowing leakage through the dam and subsequent erosion of the downstream earthen face of the dam. Also noted on the downstream slope are woodchuck holes which should be plugged and the woodchucks eradicated.

The work involved would most likely be considered ordinary maintenance and would not require the issuance of a permit by this office.

Will you please notify this office within two weeks as to your intentions in regard to this matter.

Very truly yours,

Victor F. Galgowski
Supt. of Dam Maintenance
Water & Related Resources

VFG:ljg

OFFICE OF THE MAYOR
MERIDEN, CONNECTICUT 06450

ABRAHAM G. GROSSMAN
MAYOR

May 15, 1973

Dan W. Lufkin, Commissioner
Department of Environmental Protection
State Office Building
Hartford, Connecticut 06115

Re: Bradley Hubbard Reservoir Dam - Meriden

Dear Commissioner Lufkin:

I am in receipt of your transmittal of May 10, 1973 in which you indicate that a recent inspection was made by one of your consultants relative to the subject matter.

Could you please furnish this office with the name of the consultant and his complete report so that we may make a determination as to the condition of the Dam, the extent of his recommendations relative to the work to be accomplished and any recommendations you have for carrying out the work.

The generalities which you point out in your communication cannot form a basis for the course of action that must be taken by the City of Meriden.

Upon receipt of the information requested herein, I will transmit such information to the Board of Public Works for their considerations.

Thank you for your cooperation.

Very truly yours,

Abraham G. Grossman
Mayor

FSN:cag

B-18

cc: Public Works Dept.

OFFICE OF THE MAYOR
MERIDEN, CONNECTICUT 06450

JOHN D. QUINE
MAYOR

July 8, 1974

Douglas M. Costle, Commissioner
Department of Environmental Protection
State Office Building
Hartford, Connecticut 06115

Re: Crescent Lake (Bradley and Hubbard Reservoir Dam) - Meriden

Dear Commissioner:

Please be advised that a transmittal dated May 15, 1973 from former Mayor Abraham G. Grossman to former Commissioner Lufkin has gone unanswered.

The generalities pointed out in the letter cannot form an organized basis for a course of action. Would you please furnish the City of Meriden with the following:

1. The name of the consultant who inspected the dam.

2. A complete report of the consultant's inspection and recommendations.

3. Please advise me if you are prepared to pay the cost for an engineering inspection of the dam.

4. Please advise me if you are prepared to pay for the cost of the design services.

5. Please advise me if you have any programs by which financia and technical assistance is available to make the inspectio prepare the necessary engineering documents for repair and to pay for the repairs as necessary.

It is noted in your transmittal of May 10, 1973, that the woodchuck holes should be plugged and the woodchucks eradicated

Please send me the proper procedure for plugging the woodchuck holes and eradicating the woodchucks.

Upon receipt of the information requested herein, I shall transmit such information to the Board of Public Works for their action.

Thank you for your cooperation.

Very truly yours,

John D. Quine
Mayor

JDQ:cg:N

cc: Victor Galgowski
Supt. of Dam Maintenance

COPY

STATE OF CONNECTICUT

DEPARTMENT OF ENVIRONMENTAL PROTECTION

STATE OFFICE BUILDING • HARTFORD, CONNECTICUT 06115

COPY

17 July 1974

Honorable John D. Quine
City Hall
Meriden, CT 06450

Re: Bradley and Hubbard Reservoirs
Meriden

Dear Mayor Quine:

Commissioner Costle has directed me to reply to your letter of July 8, 1974 pertaining to the subject dam.

I am enclosing a copy of our consultant's inspection report; also information on woodchuck eradication.

As indicated in my letter of May 10, 1973 to the former mayor, the required work at this site is of a maintenance nature and would not require a construction permit from our office. From the standpoint of a sound dam maintenance program the repairs are warranted.

Responsibility for maintaining dams rests with the owners of such structures. The Department of Environmental Protection does not have available funds to provide financial assistance for this type of work.

Woodchuck infestation of earthen dikes or dams is a matter that can not be treated lightly. Burrows dug into these structures can weaken the structure and lead to failure. Of the enclosed suggested methods for woodchuck eradication, we find gas bombs to be the most effective. I am sure members of your Public Works Department are familiar with this technique. The Wildlife Unit of our department will provide additional information and suggestions if needed. The person to contact is Dennis DeCarli at 566-2841.

After woodchucks have been eliminated from a dam, it is advisable to excavate around the burrows and refill the void with suitable well tamped material. An erosion-preventive cover should be provided for the disturbed surface. An alternate procedure is to fill the burrow with a concrete slurry. The important factor is to seal channels through which water could seep and eventually lead to erosion and failure of the dam.

I sincerely hope that the foregoing information will enable you to take the action necessary to place this structure in satisfactory condition. If you have further questions, please do not hesitate to call.

Very truly yours,

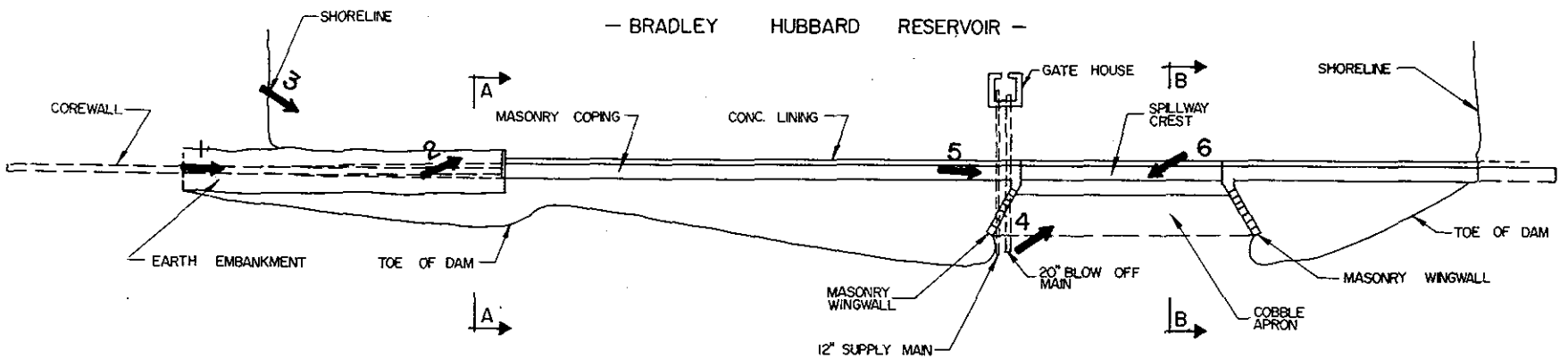
Victor F. Galgowski
Supt. of Dam Maintenance
Water & Related Resources
Telephone no. 566-7280

VFG:ljg

Enclosure

APPENDIX C

DETAIL PHOTOGRAPHS



1 ← PHOTO NUMBER AND DIRECTION

PHOTO LOCATION PLAN

BRALEY HUBBARD
RESERVOIR DAM

SHEET C-1

Photo 1 - Top of dam from right abutment. Note lack of protective cover on dike section in foreground (5/12/80)

Photo 2 - Upstream embankment of dike section (5/12/80)

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF	Bradley Hubbard Res. Dam Harbor Brook Meriden, CT
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER	INSPECTION OF NON-FED. DAMS	CE# 27 785 KE DATE Aug. '80 PAGE C-1

Photo 3 - Upstream side of masonry coping and gatehouse structure (5/12/80)

Photo 4 - Stepped masonry spillway wall. Note vegetation in spillway channel and grass growing from masonry joints. (5/12/80)

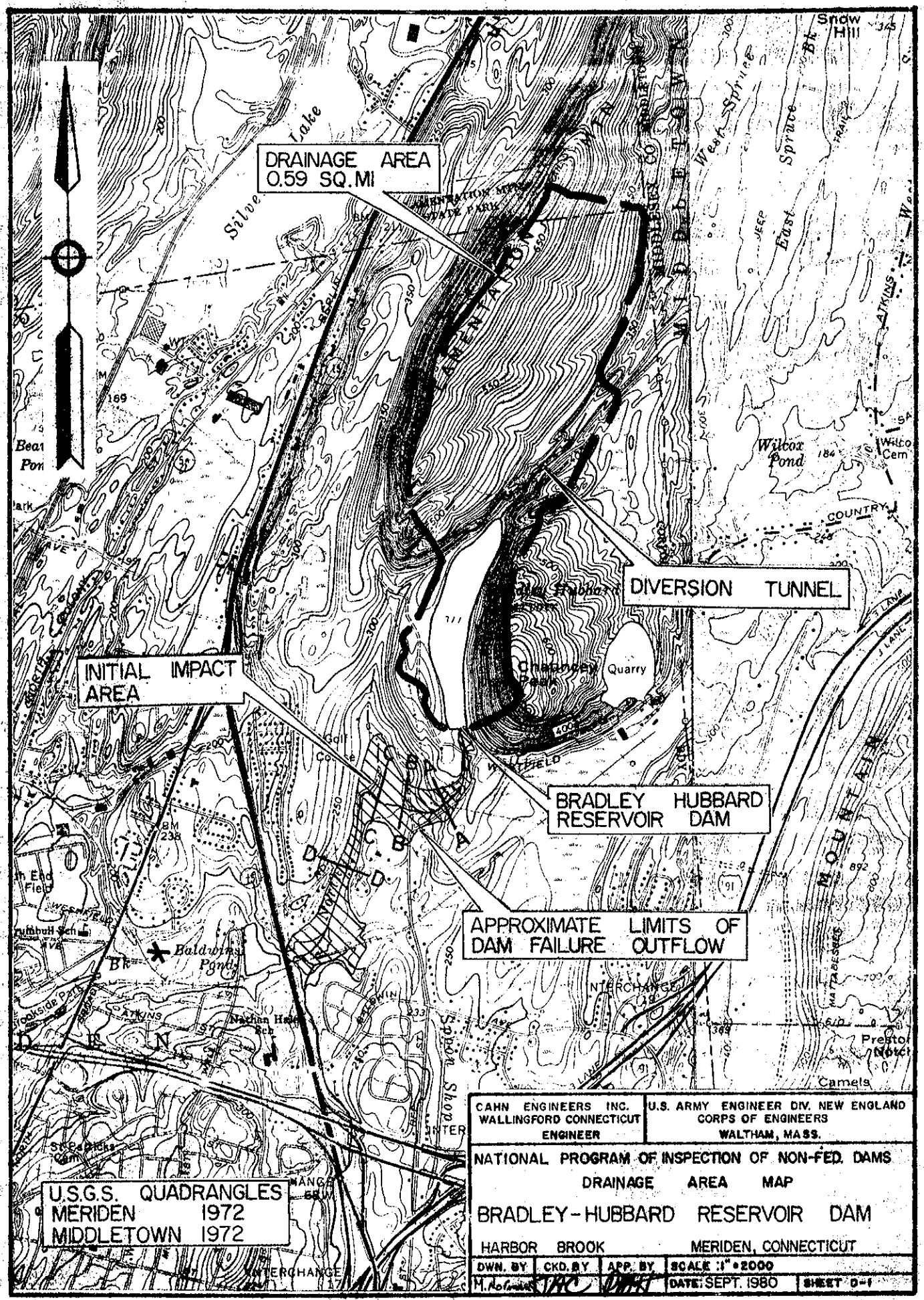
Photo 5 - Masonry spillway crest and left end of dam (5/12/80)

Photo 6 - View of spillway discharge channel from spillway crest (5/12/80)

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF	Bradley Hubbard Res. Dam Harbor Brook Meriden, CT
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER	INSPECTION OF NON-FED. DAMS	CE# 27 785 KE DATE Aug. '80 PAGE C-3

APPENDIX D

HYDRAULICS/HYDROLOGIC COMPUTATIONS


DRAINAGE AREA
0.59 SQ.MI
DIVERSION TUNNEL
INITIAL IMPACT AREA
BRADLEY HUBBARD
RESERVOIR DAM
APPROXIMATE LIMITS OF
DAM FAILURE OUTFLOW
U.S.G.S. QUADRANGLES
MERIDEN 1972
MIDDLETOWN 1972
Silver Lake
Baldwins Pond
Wilcox Pond
Chauncey Quarry
Snow Hill
West Spruce St
East Spruce St
MOUNTAINS
INTERCHANGE
CAHN ENGINEERS INC.
WALLINGFORD CONNECTICUT
ENGINEER
U.S. ARMY ENGINEER DIV. NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS
DRAINAGE AREA MAP
BRADLEY-HUBBARD RESERVOIR DAM
HARBOR BROOK
MERIDEN, CONNECTICUT
DWN. BY
CKD. BY
APP. BY
SCALE :1" = 2000
DATE: SEPT. 1980
SHEET D-1

PERFORMANCE AT PEAK FLOOD CONDITIONS

PROBABLE MAXIMUM FLOOD (PMF) DETERMINATION—

DRAINAGE AREA – 0.59 SQ.M. FROM CONN. STATE DEP BULLETIN NO. 1, 1972 (GAZETTEER OF NATURAL DRAINAGE AREAS, P-48)

WATERSHED CLASSIFICATION – "ROLLING" TO "MOUNTAINOUS" BASED ON USGS MAP AND SITE VISIT.

PMF PEAK INFLOW –
PER CORPS OF ENGINEERS GUIDANCE NOT TO EXCEED 2500 CFS/SQ. MI. FOR D.A < 2 SQ. MI, THE PEAK FLOW RATE SELECTED = 2500 CFS/SQ.M. FOR ABOVE CONDITIONS.

∴ PMF PEAK INFLOW = 2500 × 0.59 ≅ 1590 CFS.

SIZE CLASSIFICATION –
FOR THE PURPOSE OF DETERMINING PROJECT SIZE, THE MAXIMUM STORAGE ELEVATION IS CONSIDERED EQUAL TO THE TOP OF DAM
TOP OF DAM = EL. 312.0*
HEIGHT OF DAM = 16.5 FT. (FROM EXISTING DRAWINGS)

* THE W.S. ELEVATION 311' MSL ON THE MERIDEN, CT. QUADRANGLE SHEET (REV. 1972) IS ASSUMED TO BE THE SPILLWAY CREST ELEVATION ON NATIONAL GEODETIC VERTICAL DATUM (NGVD). ALL OTHER ELEVATIONS ARE REFERENCED TO THIS ASSUMED ELEVATION.

PLANIMETERING FROM USGS MAP FOR RESERVOIR SURFACE AREAS—

AT EL. 311 (SPILLWAY CREST) = 35 ACRES
AT EL. 320 = 44 ACRES
AT EL. 330 = 56 ACRES

A STAGE-RESERVOIR AREA CURVE IS PLOTTED (SHEET 3)
FROM THIS CURVE, RESERVOIR AREA AT TOP OF DAM ≅ 35.8 ACRES
AVERAGE RESERVOIR AREA BETWEEN SPILLWAY CREST
AND TOP OF DAM ≅ 35.5 Ac.
∴ STORAGE BETWEEN SPILLWAY CREST & TOP OF DAM

$= 1 \times 35.5 = 35.5$ Ac.Ft.

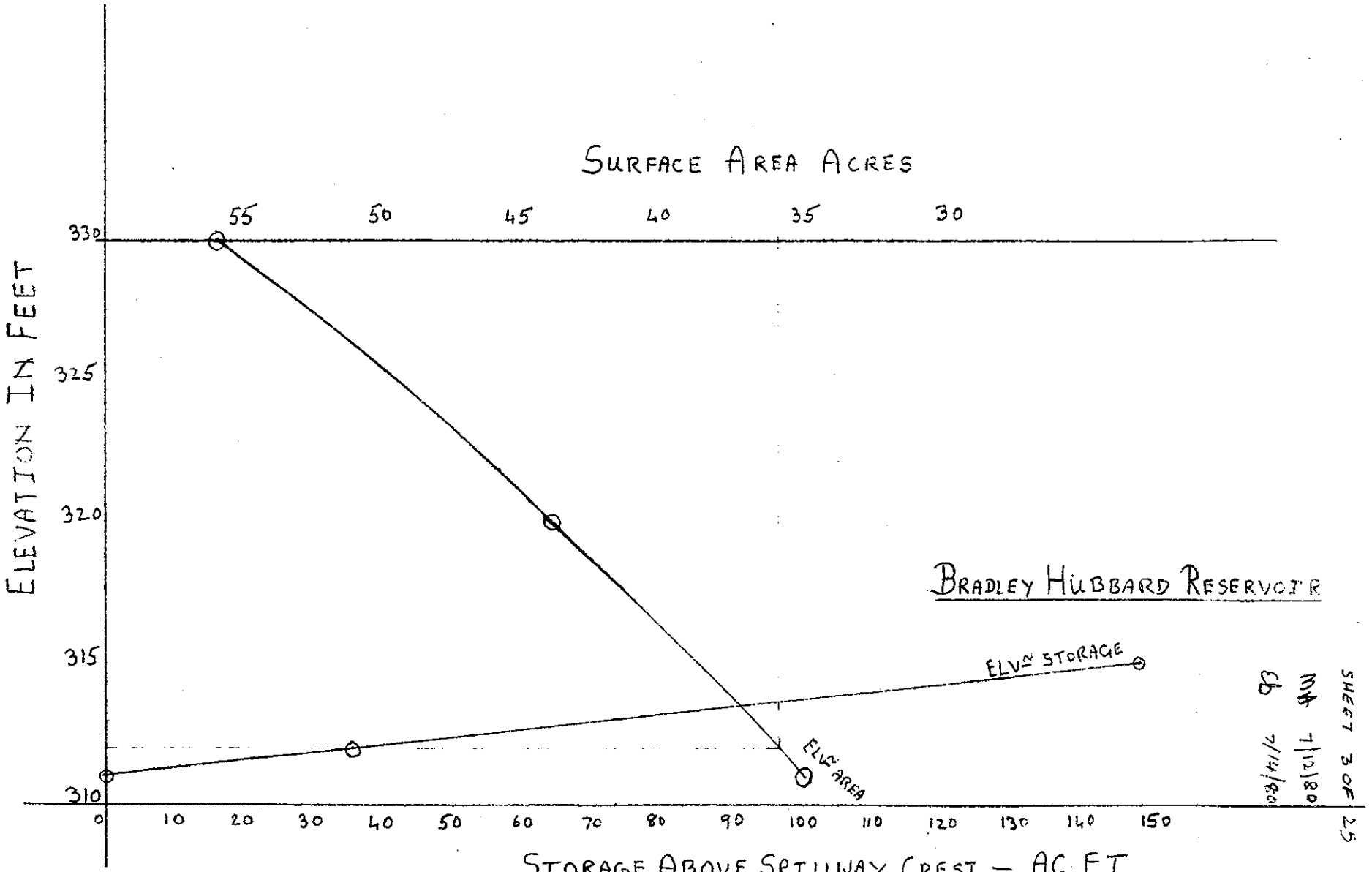
ESTIMATED STORAGE BELOW SPILLWAY CREST = $\frac{1}{3} bh$

$= \frac{1}{3} \times 35 \times 15.5 = 180$ Ac·Ft

($b = 35$ Ac·Ft, $h =$ EL 311 − EL 295.5 = 15.5')

∴ MAXIMUM IMPOUNDMENT TO TOP OF DAM = 35.5 + 180 ≅ 216 Ac.Ft.

A STAGE-STORAGE CURVE IS PLOTTED ON SHEET 3.
THUS, ACCORDING TO CORPS OF ENGINEERS GUIDELINES
TABLE 1. THE BRADLEY HUBBARD RESERVOIR DAM IS
CLASSIFIED SMALL BASED UPON THE STORAGE CAPACITY
OF 216 AC·FT. (< 1000 AND ≥ 50) AND HEIGHT OF THE
DAM IS ONLY 19.5'.



MM 7/12/80
CB 7/14/80

SHEET 3 OF 25

D-3

HAZARD POTENTIAL — HIGH HAZARD POTENTIAL DAM BASED ON DAM BREACH ANALYSIS AND RELATIVE LOCATIONS OF GOLF COURSE, HOUSES AND OTHER STRUCTURES. A DETAILED DISCUSSION OF HAZARD POTENTIAL IS INCLUDED AT THE END OF BREACH ANALYSIS SECTION OF APPENDIX -D.

SELECTION OF TEST FLOOD —
FOR THE SMALL SIZE AND HIGH HAZARD POTENTIAL CLASSIFICATION, TABLE 3 OF CORPS OF ENGINEERS RECOMMENDED GUIDELINES, THE TEST FLOOD COULD BE IN THE $\frac{1}{2}$ PMF TO PMF RANGE. BASED ON THE INVOLVED RISK POTENTIAL DOWNSTREAM OF THE DAM, A TEST FLOOD = PMF IS SELECTED (HIGH END OF THE RANGE)

TEST FLOOD PEAK INFLOW = 1500 CFS

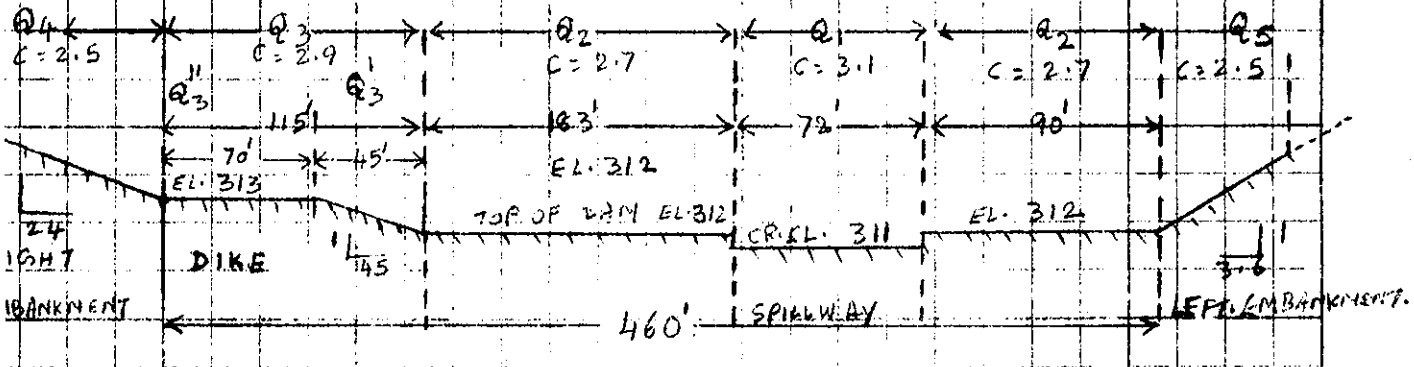
PMF WOULD RESULT FROM 19" RUN-OFF FROM 0.59 □M. OF D.A

∴ TOTAL STORM VOLUME = $\frac{19}{12} \times 0.59 \times 640 \cong 600$ AC.FT.

THUS, MAXIMUM STORAGE (BETWEEN SPILLWAY CREST AND TOP OF DAM) OF 35.5 AC·FT IS ONLY 6% OF THIS STORM VOLUME.

NOTE: SURCHARGE STORAGE ROUTING IS PERFORMED FOR $\frac{1}{2}$ PMF PEAK INFLOW ALSO

COMPOSITE DISCHARGE RATING CURVE



POTENTIAL OVERFLOW PROFILE

(BASED ON EXISTING INFORMATION & CE FIELD OBSERVATIONS)

SPILLWAY

$Q_1 = C L H^{3/2}$

$= 223 H^{3/2}$

C = 3.1 FOR BROAD CRESTED WEIR, BASED ON THE DIMENSIONS OF THE STRUCTURE, U/S & D/S SLOPES PER FIG. 7 OF USGS BOOK 3, CHAPTER A-5, "MEASUREMENT OF PEAK DISCHARGE AT DAMS BY INDIRECT METHODS"

CR. EL = 311 L = 72'

DAM

$Q_2 = C L H^{3/2}$

$= 737 H^{3/2}$

C = 2.7 FOR BROAD CRESTED WEIR.

CR. EL = 312, L = 183' + 90' = 273'

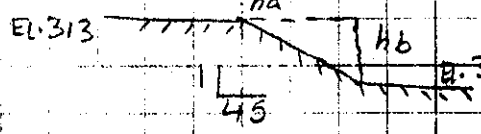
DIKE

$Q_3 = Q_3' + Q_3''$

$Q_3' = \frac{2}{5} C L \frac{(h_b^{5/2} - h_a^{5/2})}{(h_b - h_a)}$ *

C = 2.9 ASSUMED, L = 45' (APPROXIMATE)

EL = 312, h_a = 0 UP TO EL 313



*NOTE: USGS RECOMMENDED FORMULA FOR MORE PRECISE DISCHARGE OVER INCLINED DAM/EMBANKMENT CREST (REF: MEASUREMENT OF PEAK DISCHARGES AT DAMS BY INDIRECT METHODS, USGS BOOK 3, CHAPTER A.5, PAGE 3-4, 1968).

$Q_3'' = CLH^{3/2}$ C = 2.9 ASSUMED. L = 70'
$= 2.03 H^{3/2}$ CR.CL. = 3.1

SIMILARLY RIGHT EMBANKMENT Q_4 AND LEFT EMBANKMENT Q_5 ARE CALCULATED BY USCE METHOD.

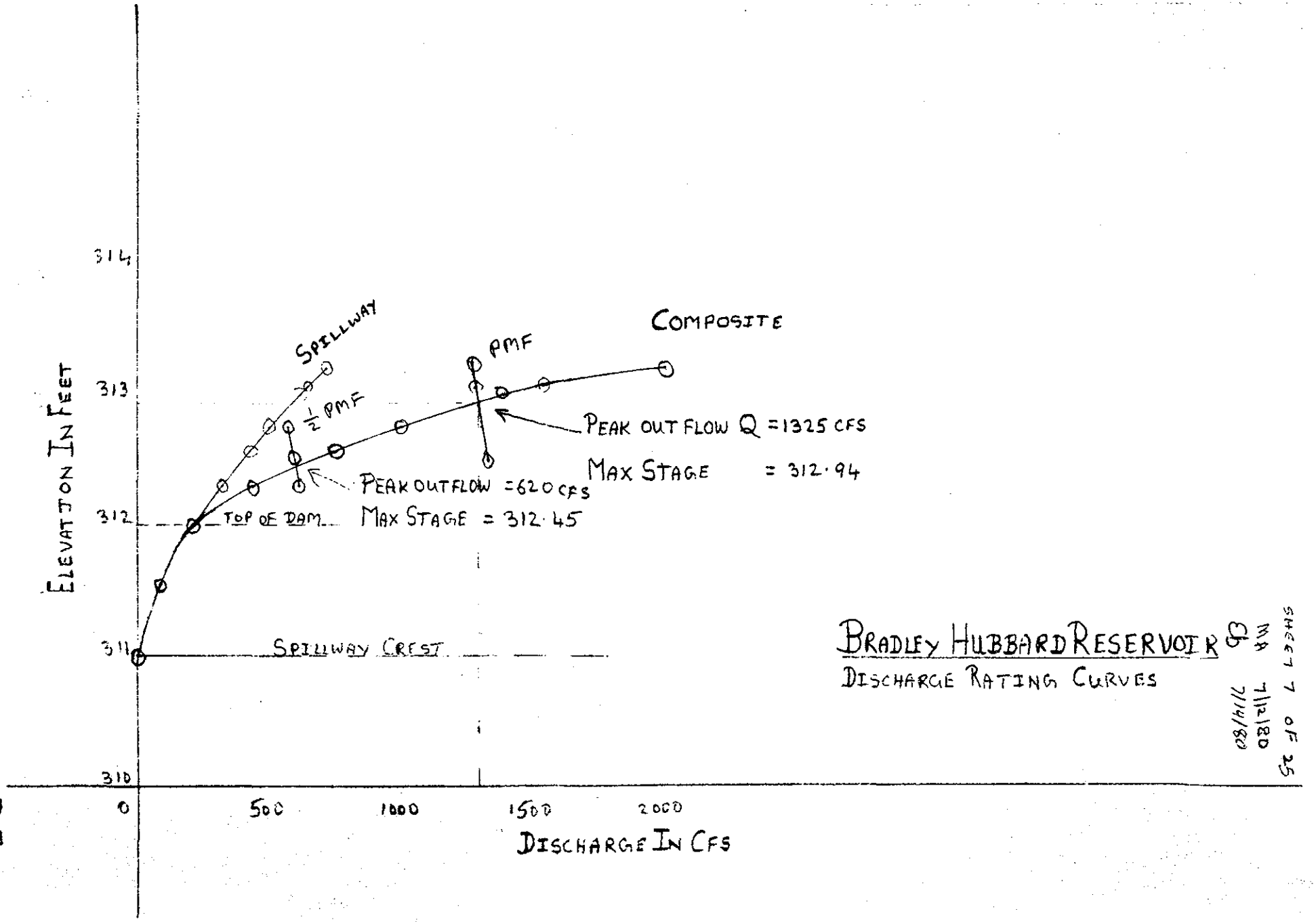
OUTLET $Q_6 = 0.6 A \sqrt{2gH}$
THE 20' DIAMETER FLOWOFF MAIN IS CONSIDERED TO BE THE LOW-LEVEL OUTLET CONDUIT AND THE DISCHARGE Q_6 FOR POOL AT TOP OF DAM IS ESTIMATED TO BE 400 CFS ACCOUNTING FOR MINOR LOSSES.

TABULATION OF DISCHARGE RATES (CFS)

ELEV	SPILLWAY Q_1	DAM Q_2	DIKE Q_3	RT. EMBANKMENT Q_4	LT EMBANKMENT, Q_5	TOTAL Q
311	0	0	0	0	0	0
311.56	93	0	0	0	0	93
TOP DAM 312	223	0	0	0	0	223
312.3	331	121	3	0	0	455
½ PMF 312.46	390	224	7	0	1	620
312.56	435	309	12	0	1	757
312.75	516	479	26	0	2	1022
TEST FLOOD 312.94	604	672	48	0	4	1325
313	631	737	52	0	4	1424
313.06	659	804	63	0	4	1530
313.2	728	959	348	0	5	2040

NOTE: CONSIDERING THE STORAGE CAPACITY ABOVE, THE DISCHARGE CAPACITY OF THE LOW LEVEL OUTLET IS NEGLECTED

DISCHARGE RATING CURVES FOR TOTAL Q (COMPOSITE) AND SPILLWAY ONLY PLOTTED ON SHEET 7.



BRADLEY HUBBARD RESERVOIR

DISCHARGE RATING CURVES

MA 7/12/80 7/14/80 SHEET 7 OF 25

D-7

DETERMINATION OF PEAK OUTFLOW—
SHORTCUT ROUTING OF RESERVOIR.
CORPS OF ENGINEERS GUIDELINES "SURCHARGE STORAGE ROUTING" ALTERNATE METHOD USED.
FOR 1500 CFS (PMF) THE DISCHARGE RATING CURVE GIVES ELVN = 313.04
AND FROM STAGE-STORAGE CURVE FOR THIS ELVN STORAGE = 73 AC-FT.

$$STOR_i = \frac{73 \times 12}{0.59 \times 640} = 2.32'' \text{ RUN-OFF}$$

$$QP_i = QP_1 \left(1 - \frac{STOR_i}{19}\right)$$

① $STOR_i$ INCHES	② $\left(1 - \frac{STOR_i}{19}\right)$	③ $STOR_i$ AC-FT ① $\times \frac{0.59 \times 640}{12}$	④ QP_i CFS ② $\times 1500$	⑤ ELVN FROM STORAGE CURVE USING ③
1.75	0.91	55	1365	312.5
2.32	0.88	73	1320	313.06
2.5	0.87	79	1305	313.24

COLUMNS ④ & ⑤ ARE PLOTTED ON DISCHARGE RATING CURVE AND

PEAK OUTFLOW @ = 1325 CFS
MAXIMUM STAGE = 312.94
TOP OF DAM = EL. 312.0

∴ THE DAM IS OVERTOPPED BY 0.94 FT.

THE ROUTING IS ALSO DONE FOR $\frac{1}{2}$ PMF

$\frac{1}{2}$ PMF PEAK INFLOW $= \frac{1}{2} \times 1500 = 750$ CFS.

DETERMINATION OF PEAK OUTFLOW-

FOR 750 CFS ($\frac{1}{2}$ PMF) THE DISCHARGE RATING CURVE GIVES
ELVN $= 312.56$
FROM STAGE-STORAGE CURVE FOR THIS ELVN = 55 AC·FT.

$STOR_i = \frac{55 \times 12}{0.59 \times 640} = 1.75$" RUN-OFF

$QP_i = QP_1 \left(1 - \frac{STOR_i}{9.5}\right)$

① $STOR_i$ INCHES	② $\left(1 - \frac{STOR_i}{9.5}\right)$	③ $STOR_i$ AC·FT ① $\times \frac{0.59 \times 640}{12}$	④ QP_i CFS ② $\times$ 750	⑤ ELVN FROM STORAGE CURVE USING ③
1.5	0.84	47	630	312.3
1.75	0.82	55	615	312.5
2.00	0.79	63	593	312.75

COLUMNS ④ & ⑤ ARE PLOTTED ON DISCHARGE RATING CURVE AND

PEAK OUTFLOW Q = 620 CFS
MAXIMUM STAGE = 312.45

TOP OF DAM = 312.0

∴ THE DAM IS OVERTOPPED BY 0.45 FT.

BREACH ANALYSIS

DOWNSTREAM FAILURE HAZARD—

BREACH OUTFLOW $Q_b = \frac{8}{27} \times W_b \times \sqrt{g} \times y_o^{3/2}$

MID-HEIGHT LENGTH = 260' FROM EXISTING DRAWINGS
W_b = 40% OF MID-HEIGHT LENGTH
W_b = 40% OF 260 = 104'.

HEIGHT OF DAM @ FAILURE y_o = 16.5' (POOL AT TOP OF DAM)
PEAK FAILURE OUTFLOW $Q_M = Q_b$ SINCE THE SPILLWAY AND LOW-LEVEL OUTLET ARE WITHIN THE BREACH WIDTH.

$\therefore Q_{P_1} = \frac{8}{27} \times 104 \times \sqrt{32.2} \times (16.5)^{3/2}$

PEAK FAILURE OUTFLOW $\cong$ 11,700 CFS

ESTIMATED FAILURE FLOOD DEPTH = $0.44\, y_o \cong 0.44 \times 16.5$
IMMEDIATELY D/S OF DAM = 7 FT.

PERFORM DOWNSTREAM ROUTING OF PEAK FAILURE OUTFLOW
SELECT A SECTION AA 1350' D/S OF THE DAM

USING MANNING'S EQUATION

$Q = \frac{1.486}{n} \times A \times R^{2/3} \times S^{\frac{1}{2}}$ WHERE n = 0.07 ASSUMED (BRUSH, STONES) BASED ON CHAPTER 5 "OPEN CHANNEL HYDRAULICS" BY VEN TE CHOW

$= 4.4 \times A \times R^{2/3}$

S = 0.043 ESTIMATED FROM USGS MAP

ELVN	A, SQ.FT	P	R = A/P	$R^{2/3}$	Q CFS
260	0	—	—	—	—
265	250	101	2.48	1.83	2000
268	640	161	4	2.5	7040
270	1000	201	5	2.92	12,850

STAGE AREA AND STAGE DISCHARGE CURVES ARE PLOTTED FOR SECTION AA

FOR PEAK FAILURE OUTFLOW QP_1 = 11,700 CFS, ELVN = 269.8

FROM STAGE DISCHARGE CURVE, AND STAGE AREA CURVE GIVES AREA = 970 SQ.FT.

VOLUME OF REACH $V_1 = \frac{1350 \times 970}{43,560} = 30$ AC.FT.

TRIAL $QP_2 = QP_1 \left(1 - \frac{V_1}{S}\right)$, WHERE S = TOTAL STORAGE TO TOP OF DAM = 246 AC.FT.

$= 11,700 \left(1 - \frac{30}{246}\right) \cong 10,100$ CFS

FOR THIS QP_2 THE STAGE DISCHARGE CURVE GIVES

ELVN = 269.6

AND AREA = 922 SQ.FT.

$\therefore V_2 = \frac{1350 \times 922}{43,560} \cong 29$ AC.FT.

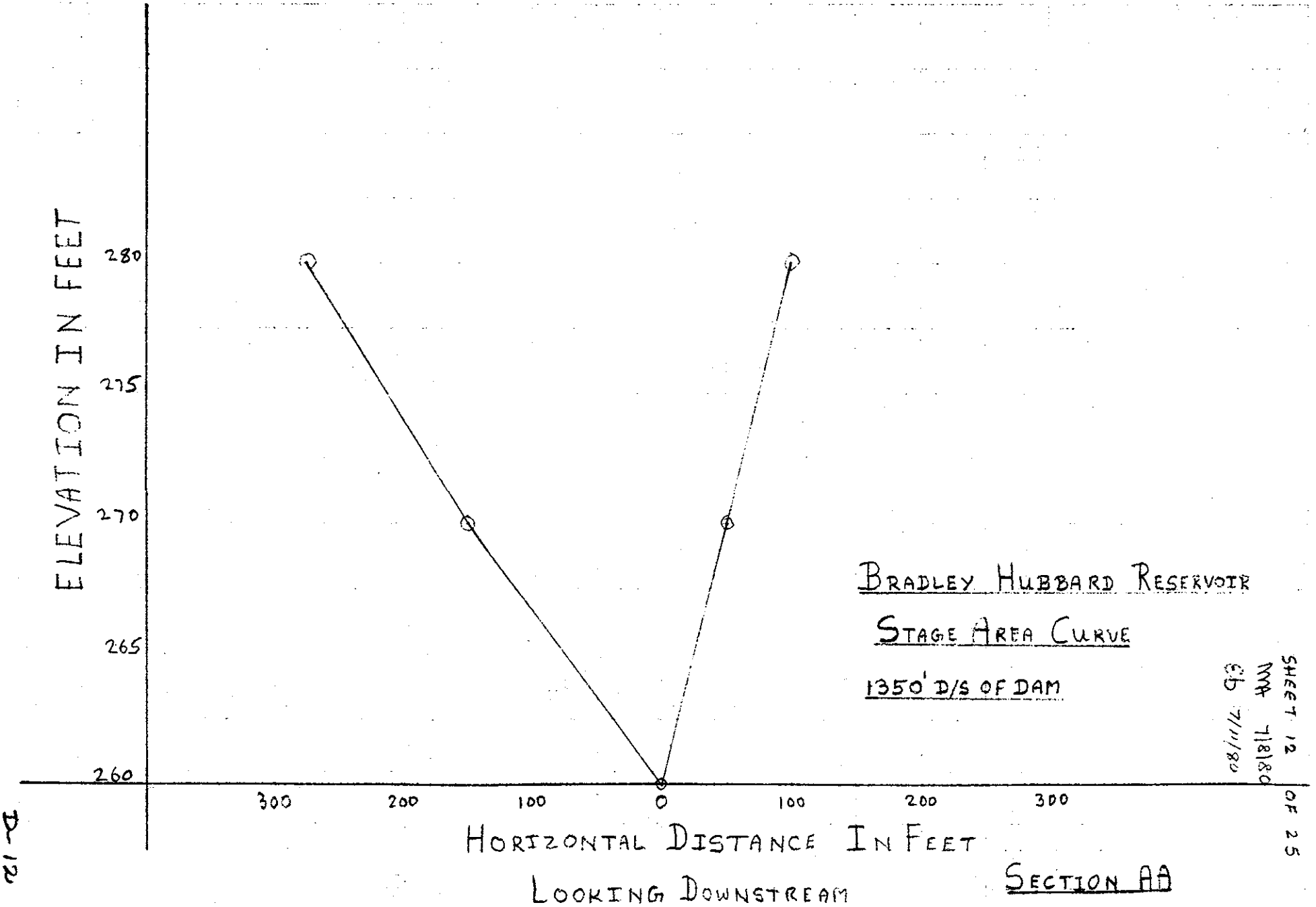
RECOMPUTING $QP_2 = 11,700 \left(1 - \frac{30+29}{246}\right) = 10,100$ CFS

AND FLOOD STAGE AT SECTION AA = 269.6

FLOOD DEPTH AT SECTION AA = El.269.6 - El.260

= 9.6 FT.

AND VELOCITY AT SECTION AA $= \frac{10,100}{922} \cong 11$ FPS



BRADLEY HUBBARD RESERVOIR

STAGE AREA CURVE

1350' D/S OF DAM

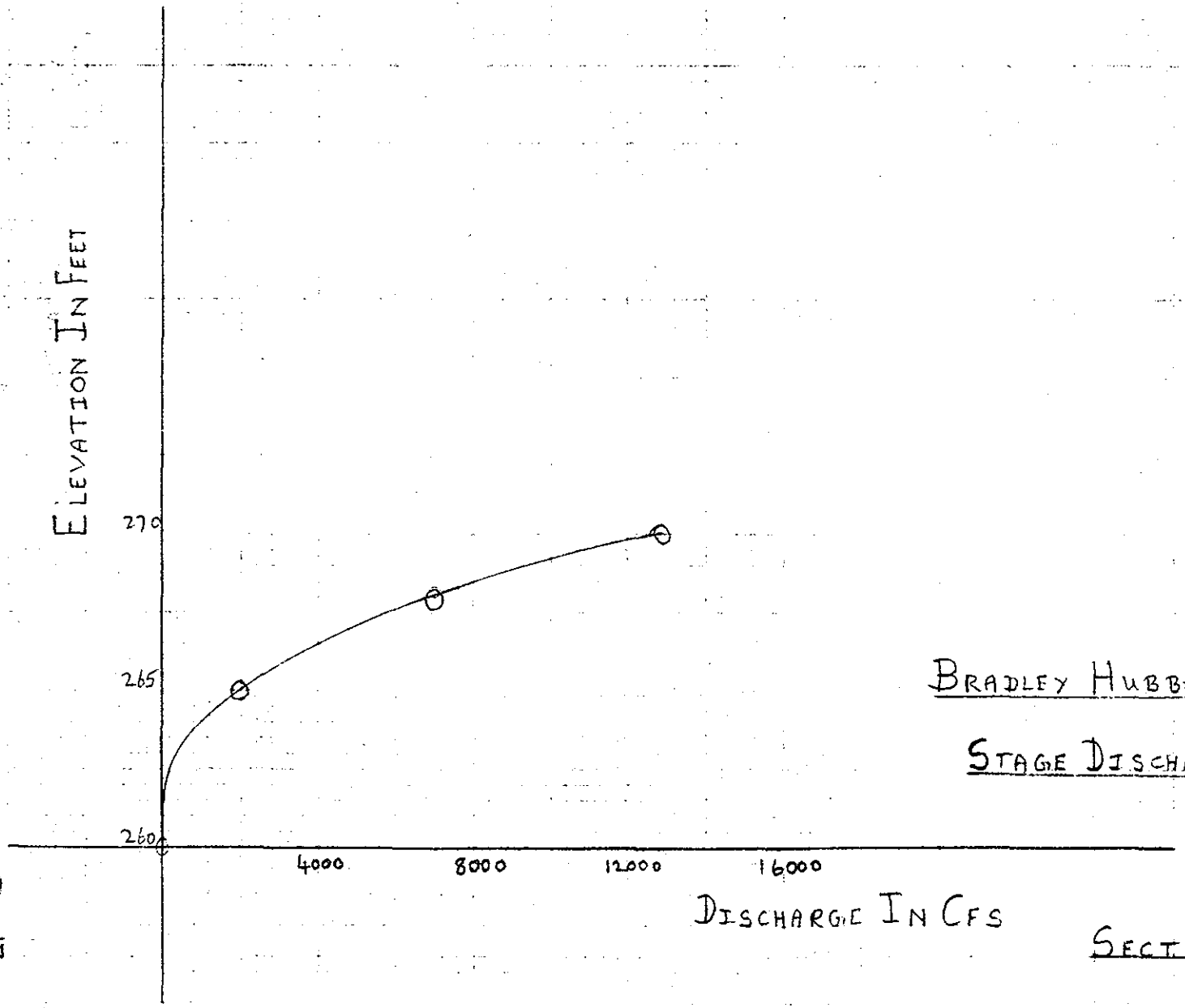
SECTION AA

MM 7/18/80

GB 7/11/80

SHEET 12 OF 25

D-12



BRADLEY HUBBARD RESERVOIR

STAGE DISCHARGE CURVE

SECTION AA

08/11/80 MA 71880 SHEET 13 OF 25

SELECTING A SECTION BB 600 FT D/S OF SECTION AA

$Q = \frac{1.486}{n} \times A \times R^{2/3} \times S^{1/2}$ WHERE $n = 0.07$ ASSUMED

$= 5.5 \times A \times R^{2/3}$ $S = 0.067$ ESTIMATED FROM USGS MAP

EL	A, SQ FT	P	R = A/P	$R^{2/3}$	Q CFS
220	0	–	–	–	–
225	500	200	2.5	1.84	5,100
227	980	280	3.5	2.30	12,400

STAGE AREA AND STAGE DISCHARGE CURVES ARE PLOTTED FOR $QP_1 = 10,100$ CFS, ELVN = 226.8 AND FROM STAGE CURVE, AREA = 918 SQ. FT.

VOLUME OF REACH $V_1 = \frac{600 \times 918}{43,560} = 13$ AC. FT.

STORAGE REMAINING $= 216 - \frac{30+29}{2} \cong 186$ AC. FT.

TRIAL $QP_2 = QP_1 \left(1 - \frac{V_1}{S}\right) = 10,100 \left(1 - \frac{13}{186}\right) = 9400$ CFS

FOR 9400 CFS, ELVN = 226.6 AND AREA = 871 SQ. FT

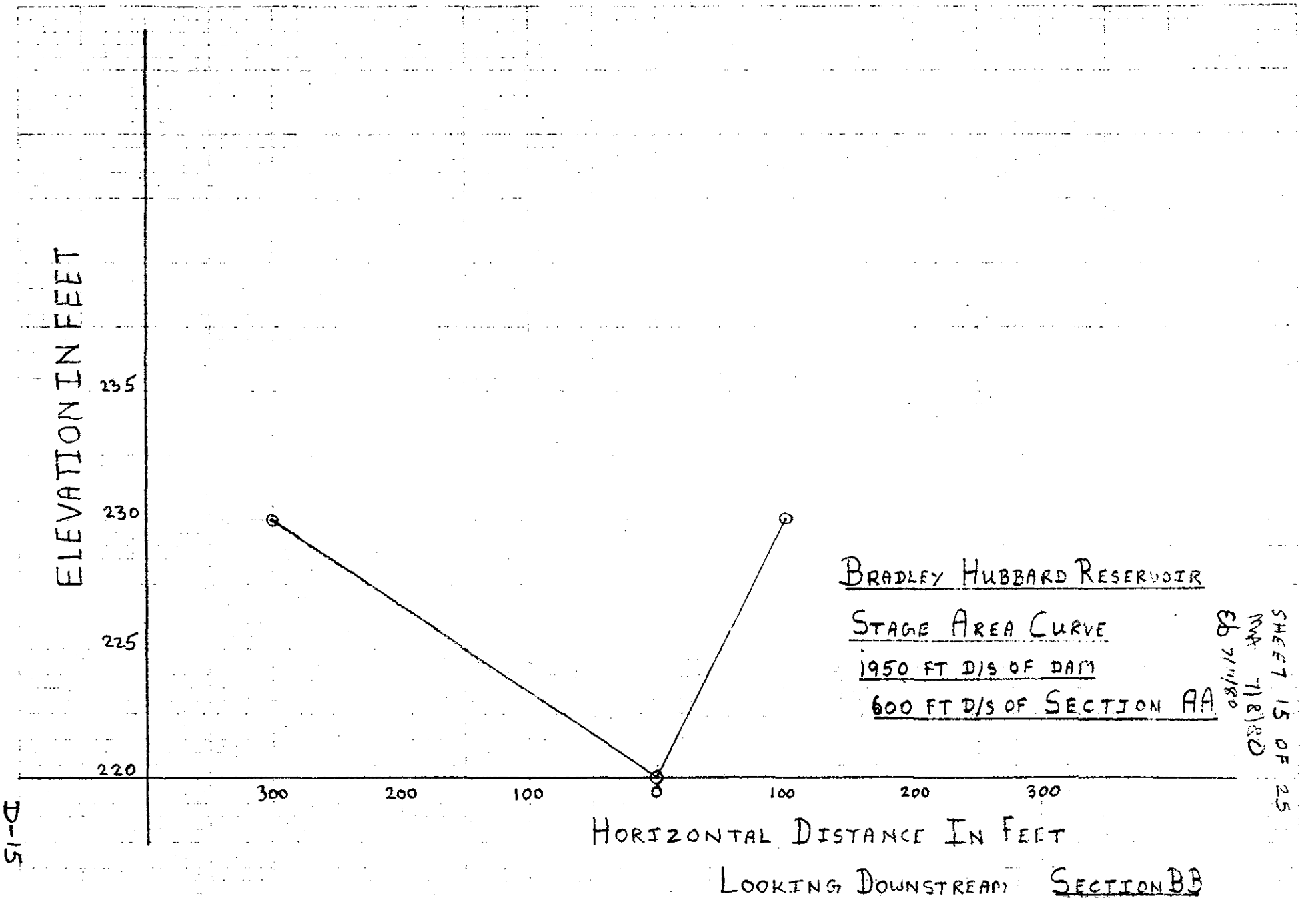
$V_2 = \frac{600 \times 871}{43,560} = 12$ AC. FT.

RECOMPUTING $QP_2 = 10,100 \left(1 - \frac{\frac{13+12}{2}}{186}\right) \cong 9,400$ CFS

AND FLOOD STAGE AT SECTION BB = 226.6

DEPTH OF FLOOD WATER AT SECTION BB = EL 226.6 – EL 220
= 6.6 FT.

VELOCITY AT SECTION BB = $\frac{9400}{871} \cong 11$ FPS



BRADLEY HUBBARD RESERVOIR

STAGE AREA CURVE

1950 FT D/S OF DAM

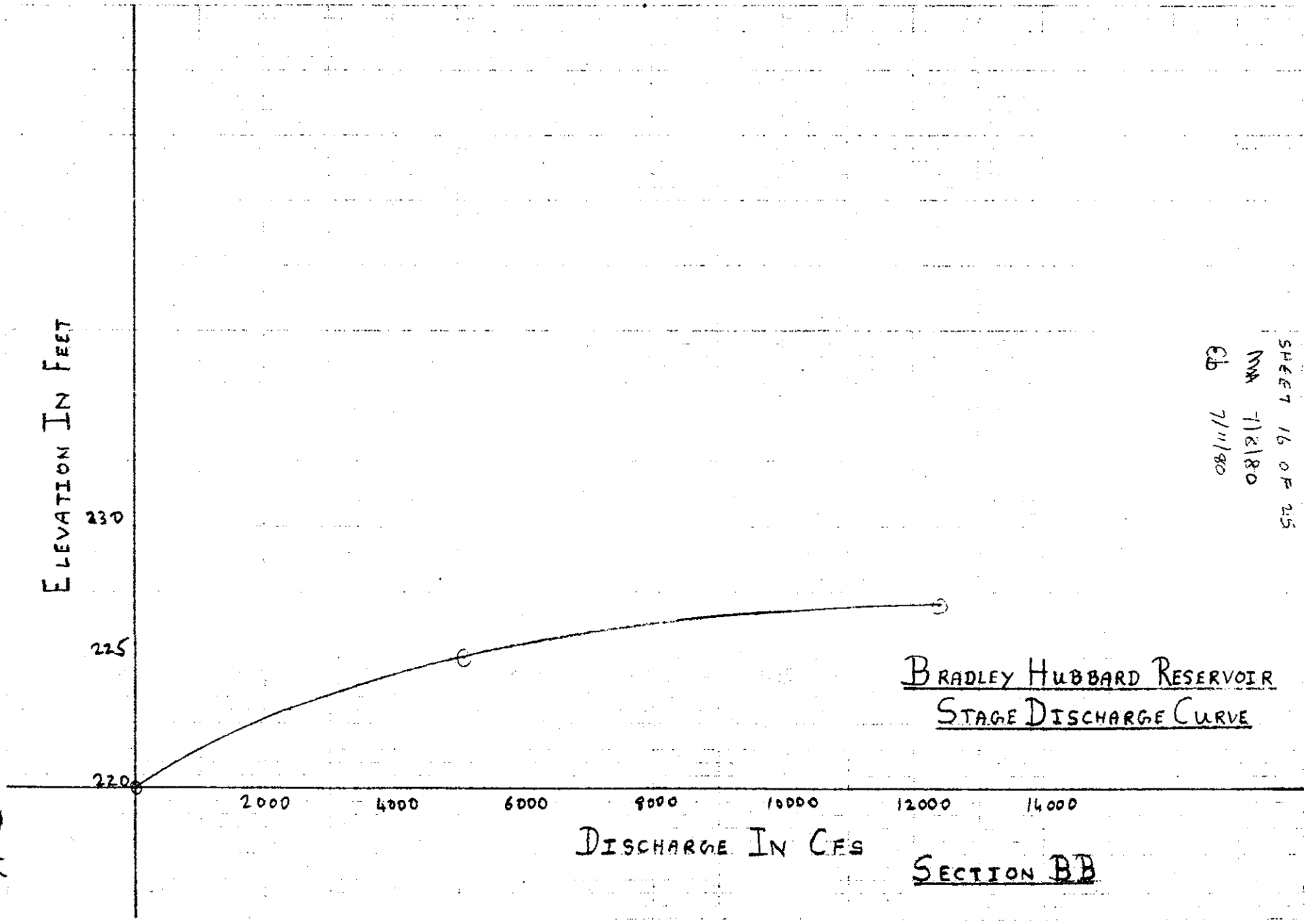
600 FT D/S OF SECTION AA

SECTION BB

BY MWA 7/18/80

SHEET 15 OF 25

A-15



BRADLEY HUBBARD RESERVOIR
STAGE DISCHARGE CURVE

SECTION BB

SHEET 16 OF 25
MM 7/28/80
CB 7/11/80

D-16

SELECT A SECTION CC 500' D/S OF SECTION BB

$Q = \frac{1.486}{n} \times A \times R^{2/3} \times S^{1/2}$

$= 4.00 \times A \times R^{2/3}$

WHERE n = 0.05 ASSUMED (Maintained stream section, Windy-f)

S = 0.018 ESTIMATED FROM USGS MAP

ELVN	A SQ. FT	P	R = A/P	$R^{2/3}$	Q CFS
211	0	—	—	—	—
215	1072	536	2	1.6	6.900
217	2370	790	3	2.1	19.900

STAGE AREA AND STAGE DISCHARGE CURVES ARE PLOTTED.
FOR $Q P_1$ = 9,400 CFS. ELVN = 215.8 AND AREA = 1524 SQ FT.

VOLUME OF REACH $V_1 = \frac{500 \times 1524}{43,560} \cong 18$ AC.FT.

STORAGE REMAINING = $186 - \frac{13+12}{2} = 174$ AC.FT

TRIAL $Q P_2 = Q P_1 \left(1 - \frac{V_1}{S}\right)$

$= 9400 \left(1 - \frac{18}{174}\right) \cong 8,400$ CFS

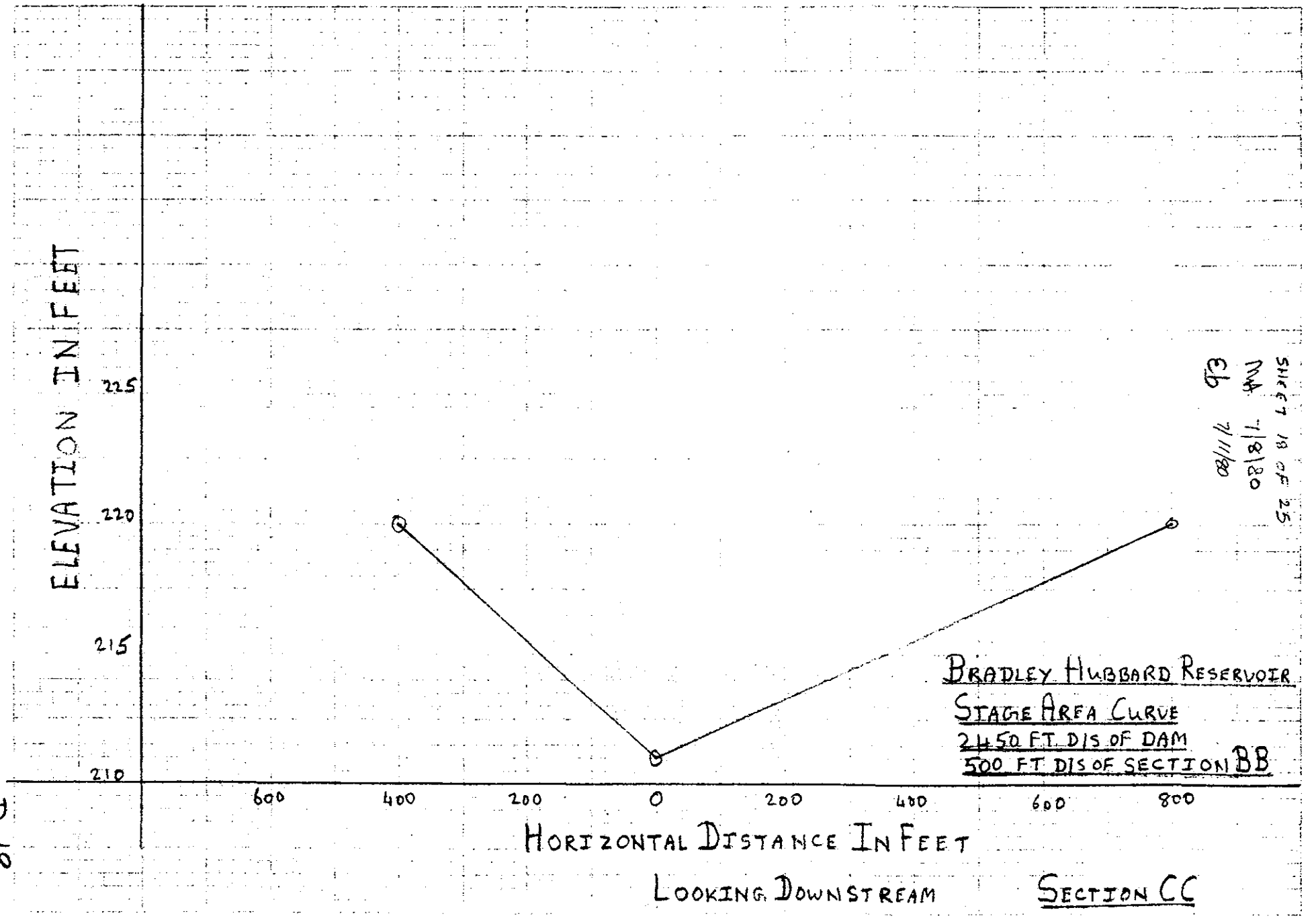
FOR 8,400 CFS - ELVN = 215.5 AND AREA = 1350 SQ FT.

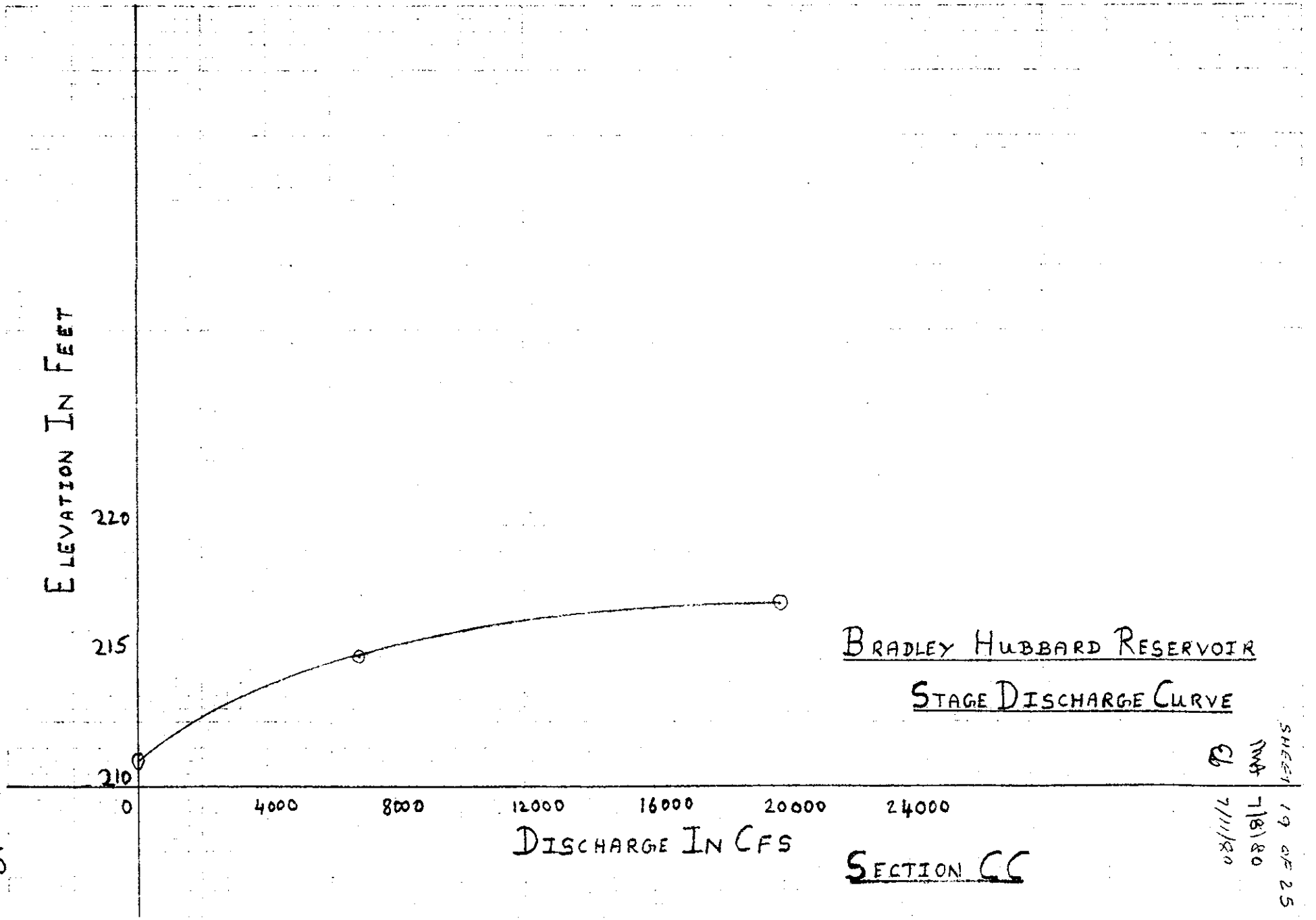
$V_2 = \frac{500 \times 1350}{43,560} = 16$ AC. FT.

RECOMPUTING $Q P_2 = 9,400 \left(1 - \frac{18+16}{2} / 174\right) \cong 8,500$ CFS
AND FLOOD STAGE = 215.5

DEPTH OF FLOOD WATER AT SECTION CC = EL. 215.5 − EL. 211
= 4.5 FT.

VELOCITY AT SECTION CC = $\frac{8,500}{1350} \cong 6$ FPS


ELEVATION IN FEET
225
220
215
210
600
400
200
0
200
400
600
800
HORIZONTAL DISTANCE IN FEET
LOOKING DOWNSTREAM
SECTION CC
BRADLEY HUBBARD RESERVOIR
STAGE AREA CURVE
2450 FT. DIS OF DAM
500 FT DIS OF SECTION BB
SHEET 18 OF 25
MM 7/18/80
8/11/80



BRADLEY HUBBARD RESERVOIR

STAGE DISCHARGE CURVE

SECTION CC

CB
7/18/80
7/11/80

SHEET 19 OF 25

D-19

SELECT A SECTION DD 1350 FT D/S OF SECTION CC

$Q = \frac{1.486}{n} \times A \times R^{2/3} \times S^{1/2}$ WHERE $n = 0.05$ ASSUMED

$= 1.8 \times A \times R^{2/3}$ $S = 0.0037$ ESTIMATED FROM USGS MAP

EL	A SQ.FT	P	R = A/P	$R^{2/3}$	Q CFS
205	0	—	—	—	—
210	188	76	2.47	1.83	600
215	1000	256	4.00	2.52	4500
218	1908	356	5.36	3.06	10,300

STAGE AREA AND STAGE DISCHARGE CURVES ARE PLOTTED FOR $Q P_1 = 8,500$ CFS. ELVN = 217.4 AND AREA = 1700 SQ.FT.

VOLUME OF REACH = $\frac{1350 \times 1700}{43,560} = 53$ AC.FT.

STORAGE REMAINING = $174 - \frac{18+16}{2} = 157$ AC.FT.

TRIAL $Q P_2 = Q P_1 \left(1 - \frac{V_1}{S}\right) = 8,500 \left(1 - \frac{53}{157}\right) = 5,600$ CFS.

FOR 5,600 CFS. ELVN = 215.8 AND AREA = 1218 SQ.FT.

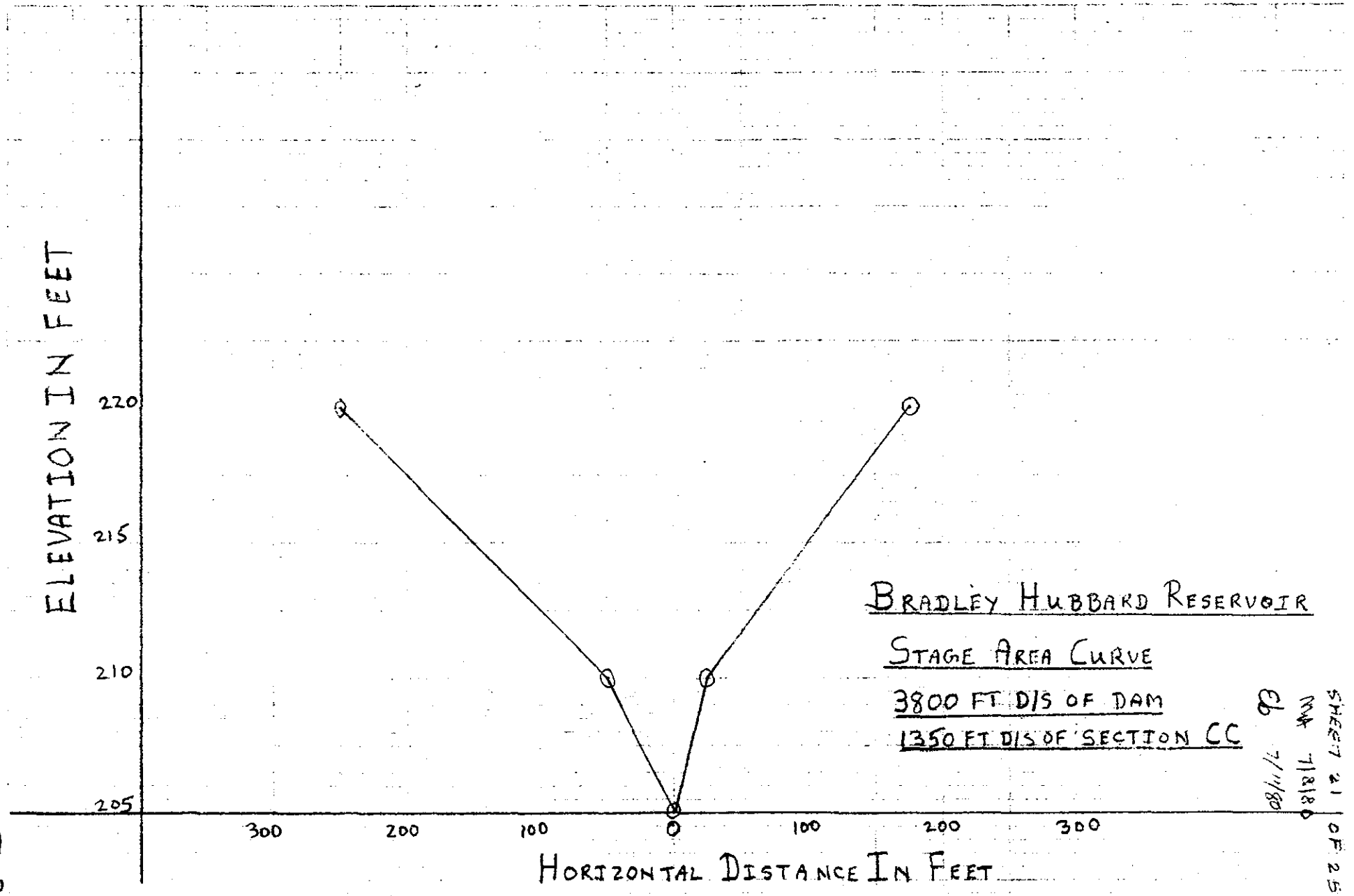
$V_2 = \frac{1350 \times 1218}{43,560} = 38$ AC.FT

RECOMPUTING $Q P_2 = 8,500 \left(1 - \frac{23+38}{157}\right) \simeq 6,000$ CFS

AND FLOOD STAGE = 216

DEPTH OF FLOOD WATER AT SECTION DD = EL. 216 - EL. 205 = 11 FT.

VELOCITY AT SECTION DD = $\frac{6,000}{1268} \simeq$ 5 FPS



BRADLEY HUBBARD RESERVOIR

STAGE AREA CURVE

3800 FT D/S OF DAM

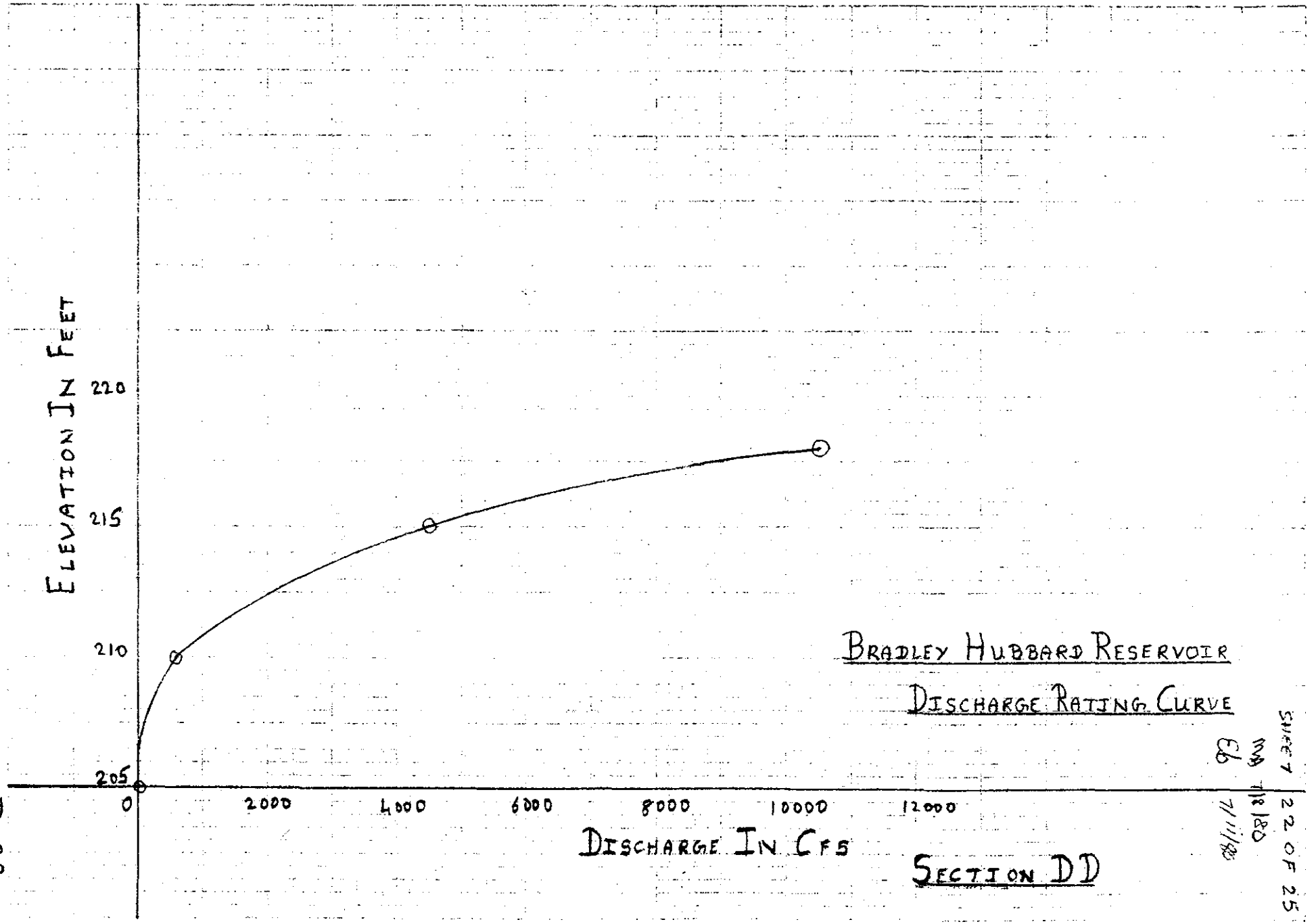
1350 FT D/S OF SECTION CC

CB 7/11/80

MA 7/18/80

SHEET 21 OF 25

D-21



BRADLEY HUBBARD RESERVOIR

DISCHARGE RATING CURVE

SECTION DD

SHEET 22 OF 25

MA 7/11/80

EB 7/11/80

D-22

FAILURE HAZARD POTENTIAL

SUMMARY OF BREACH ANALYSIS RESULTS—

LOCATION	DISTANCE FROM DAM - FT.	PEAK FLOW RATE CFS	FLOOD STAGE	FLOOD DEPTH FT.	VELOCITY FPS	STORAGE VOL. REMAINING AC-FT.
DAM	0	11,700	302.5	7	–	216
AA	1350	10,100	269.6	9.6	11	186
BB	1950	9,400	226.6	6.6	11	174
CC	2450	8,500	215.5	4.5	6	157
DD	3800	6,000	216	11	5	112

A FLOOD OF THIS MAGNITUDE WOULD IMPACT GEORGE HUNTER GOLF COURSE, AT LEAST TWO HOMES, WESTFIELD ROAD AND THREE CULVERTS DOWNSTREAM. THE SERIOUSNESS OF THE IMPACT IS DISCUSSED BELOW:

THE DEPTH OF FLOOD WATER AT DAM FAILURE IS ESTIMATED TO BE IN THE RANGE OF 7 FT. TO 9.6 FT BETWEEN THE DAM AND SECTION AA 1350 FT. DOWNSTREAM WITH VELOCITIES IN THE 11± FPS RANGE, AND WOULD THEREFORE DAMAGE THE CULVERT LOCATED 400± FT. BELOW THE DAM AT AN ACCESS ROAD TO THE DAM SITE AS WELL AS INUNDATE WESTFIELD ROAD.

SECTION BB IS TAKEN AT THE EASTERN EDGE OF THE GOLF COURSE ADJACENT TO A CULVERT. AT THIS SECTION THE FLOOD DEPTH IS ESTIMATED TO BE 6.6 FT. WITH A VELOCITY OF 11 FPS; HENCE THE CULVERT AND THE ROAD WOULD BE DAMAGED.

SECTION CC IS TAKEN AT THE CENTER OF THE GOLF COURSE, WHERE THE FLOOD DEPTH IS ESTIMATED TO BE 4.5 FEET WITH A VELOCITY OF 6 FPS AND AT SECTION DD TAKEN 100± FT FROM THE SOUTHERN EDGE OF THE GOLF COURSE ADJACENT TO A HOME ON WESTFIELD ROAD, THE FLOOD

DEPTH IS ESTIMATED TO BE 11FT. WITH A VELOCITY OF 5 FPS. THUS, A SIGNIFICANT PORTION OF THE GOLF COURSE WOULD BE INUNDATED WITH FLOOD WATER. THIS ACTIVE GOLF COURSE IS CONSIDERED AS INITIAL IMPACT AREA.

THE HOUSE NORTH OF WESTFIELD RD. AND ADJACENT TO SECTION DD HAS A 1ST FLOOR ELEVATION OF 7.6 ± FT. ABOVE CHANNEL BED AND THEREFORE WOULD BE INUNDATED WITH 3.4± FT. OF WATER. SIMILARLY, THE HOUSE LOCATED SOUTH OF WESTFIELD RD WOULD BE INUNDATED WITH 2.2± FT. OF WATER, SINCE IT'S 1ST FLOOR ELEVATION IS 8.8± FT. ABOVE CHANNEL BED. IN ADDITION, A PORTION OF WESTFIELD ROAD WOULD BE INUNDATED WITH 2.5± FT. OF WATER. THESE TWO HOMES AND WESTFIELD RD. ARE CONSIDERED SECONDARY IMPACT AREA.

AT THE END OF FLOOD ROUTING ANALYSIS, 112 AC.FT. OF STORAGE VOLUME IS REMAINING AND ONLY 48% OF THE TOTAL STORAGE VOLUME HAS BEEN ATTENUATED. THUS, THE REACH FURTHER DOWNSTREAM INCLUDING BALDWIN'S POND COULD BE IMPACTED AND IT IS SUGGESTED THAT THIS POTENTIAL IMPACT AREA SHOULD BE INCLUDED IN A FUTURE PHASE II INVESTIGATION.

BASED ON THE ABOVE ANALYSIS, A HAZARD POTENTIAL OF HIGH MAGNITUDE IS CONSIDERED LIKELY.

SUMMARY- HYDRAULIC/HYDROLOGIC COMPUTATIONS

TEST FLOOD PEAK INFLOW PMF 1500 CFS
(PARALLEL COMPUTATIONS HAVE BEEN PERFORMED FOR ½PMF PEAK INFLOW AND RESULTS ARE SUMMARIZED BELOW)

PERFORMANCE AT PEAK FLOOD CONDITIONS:	PMF	½PMF
PEAK INFLOWS CFS	1500	750
PEAK OUTFLOWS CFS	1325	620
SPILL.CAP. TO TOP OF DAM (EL.312 NGVD) CFS	223	223
SPILL.CAP. TO TOP OF DAM % OF PEAK OUTFLOW	17	36
SPILL. CAP. TO PEAK FLOOD ELEVN. CFS	604	390
SPILL. CAP. TO PEAK FLOOD ELVN. % OF PEAK OUTFLOW	46	63
PERFORMANCE:		
MAXIMUM POOL ELEVN NGVD	312.94	312.45
MAX. SURCHARGE HEIGHT ABOVE SPILL.CREST FT.	1.94	1.45
DAM OVERTOPPED FT.	0.94	0.45

DOWNSTREAM FAILURE CONDITIONS:

PEAK FAILURE OUTFLOW CFS 11,700
FLOOD DEPTH IMMEDIATELY D/S FROM DAM 7 FT
CONDITIONS AT THE INITIAL IMPACT AREA (MIDDLE OF GOLF COURSE AT CC)
THE CONDITIONS VARY FROM SECTION BB TO SECTION DD.
ESTIMATED STAGE BEFORE FAILURE WITH 223 CFS 211.2NGVD
ESTIMATED STAGE AFTER FAILURE WITH 8,500 CFS 215.5NGVD
ESTIMATED RAISE IN STAGE AFTER FAILURE ΔY_1 4.3FT
CONDITIONS AT THE SECONDARY IMPACT AREA:
ESTIMATED STAGE BEFORE FAILURE WITH 223 CFS (AT SECTION DD) 208.1NGVD
ESTIMATED STAGE AFTER FAILURE WITH 6000 CFS 216. NGVD
ESTIMATED RAISE IN STAGE AFTER FAILURE ΔY_2 7.9FT

PRELIMINARY GUIDANCE

FOR ESTIMATING

MAXIMUM PROBABLE DISCHARGES

IN

PHASE I DAM SAFETY

INVESTIGATIONS

New England Division
Corps of Engineers

March 1978

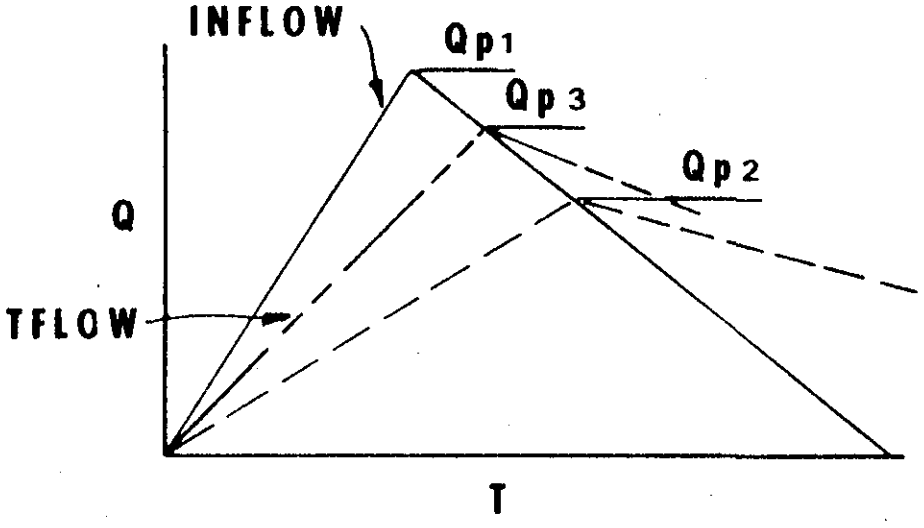
MAXIMUM PROBABLE FLOOD INFLOWS
NED RESERVOIRS

	Project	Q (cfs)	D.A. (sq. mi.)	MPF cfs/sq. mi.
1.	Hall Meadow Brook	26,600	17.2	1,546
2.	East Branch	15,500	9.25	1,675
3.	Thomaston	158,000	97.2	1,625
4.	Northfield Brook	9,000	5.7	1,580
5.	Black Rock	35,000	20.4	1,715
6.	Hancock Brook	20,700	12.0	1,725
7.	Hop Brook	26,400	16.4	1,610
8.	Tully	47,000	50.0	940
9.	Barre Falls	61,000	55.0	1,109
10.	Conant Brook	11,900	7.8	1,525
11.	Knightville	160,000	162.0	987
12.	Littleville	98,000	52.3	1,870
13.	Colebrook River	165,000	118.0	1,400
14.	Mad River	30,000	18.2	1,650
15.	Sucker Brook	6,500	3.43	1,895
16.	Union Village	110,000	126.0	873
17.	North Hartland	199,000	220.0	904
18.	North Springfield	157,000	158.0	994
19.	Ball Mountain	190,000	172.0	1,105
20.	Townshend	228,000	106.0(278 total)	820
21.	Surry Mountain	63,000	100.0	630
22.	Otter Brook	45,000	47.0	957
23.	Birch Hill	88,500	175.0	505
24.	East Brimfield	73,900	67.5	1,095
25.	Westville	38,400	99.5(32 net)	1,200
26.	West Thompson	85,000	173.5(74 net)	1,150
27.	Hodges Village	35,600	31.1	1,145
28.	Buffumville	36,500	26.5	1,377
29.	Mansfield Hollow	125,000	159.0	786
30.	West Hill	26,000	28.0	928
31.	Franklin Falls	210,000	1000.0	210
32.	Blackwater	66,500	128.0	520
33.	Hopkinton	135,000	426.0	316
34.	Everett	68,000	64.0	1,062
35.	MacDowell	36,300	44.0	825

MAXIMUM PROBABLE FLOWS
BASED ON TWICE THE
STANDARD PROJECT FLOOD
(Flat and Coastal Areas)

	River	SPF (cfs)	D.A. (sq. mi.)	MPF (cfs/sq. mi.)
1.	Pawtuxet River	19,000	200	190
2.	Mill River (R.I.)	8,500	34	500
3.	Peters River (R.I.)	3,200	13	490
4.	Kettle Brook	8,000	30	530
5.	Sudbury River.	11,700	86	270
6.	Indian Brook (Hopk.)	1,000	5.9	340
7.	Charles River.	6,000	184	65
8.	Blackstone River.	43,000	416	200
9.	Quinebaug River	55,000	331	330

ESTIMATING EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES



STEP 1: Determine Peak Inflow (Qp_1) from Guide Curves.

STEP 2: a. Determine Surcharge Height To Pass "Qp_1".

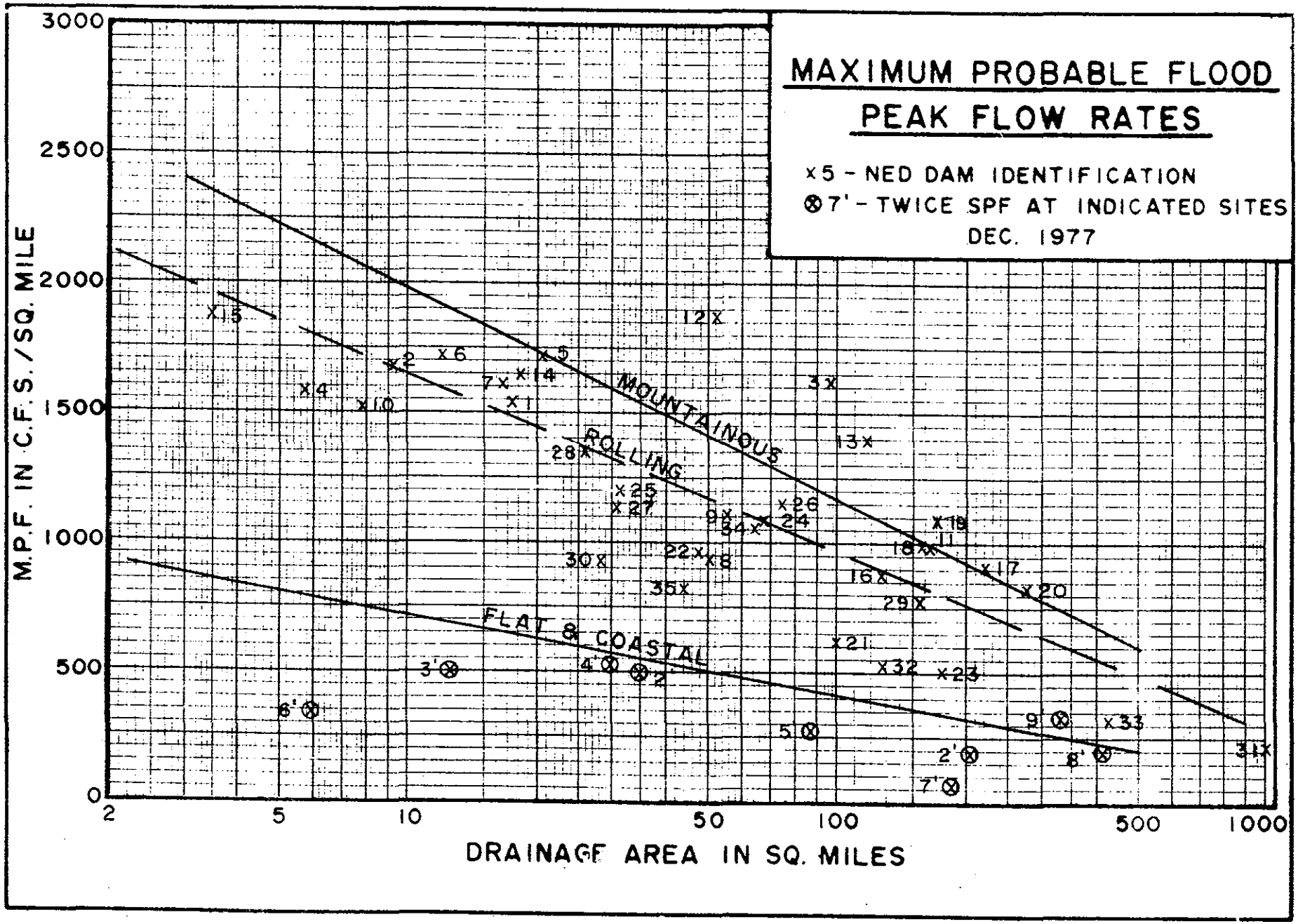
b. Determine Volume of Surcharge ($STOR_1$) In Inches of Runoff.

c. Maximum Probable Flood Runoff In New England equals Approx. 19", Therefore:

$$Qp_2 = Qp_1 \times \left(1 - \frac{STOR_1}{19}\right)$$

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Qp_2"

b. Average "$STOR_1$" and "$STOR_2$" and Determine Average Surcharge and Resulting Peak Outflow "Qp_3".


MAXIMUM PROBABLE FLOOD
PEAK FLOW RATES
x5 - NED DAM IDENTIFICATION
⊗7' - TWICE SPF AT INDICATED SITES
DEC. 1977
M.P.F. IN C.F.S./SQ. MILE
DRAINAGE AREA IN SQ. MILES
3000
2500
2000
1500
1000
500
0
2
5
10
50
100
500
1000
MOUNTAINOUS
ROLLING
FLAT & COASTAL
x15
x4
x10
x2
x6
7x
x14
x1
x5
12x
3x
13x
28x
x25
x27
x26
24
9x
54x
x18
11
18x
x17
x20
16x
29x
30x
22x
x8
35x
x21
x32
x23
3'⊗
4'⊗
⊗2
6'⊗
5'⊗
9'⊗
x33
2'⊗
8'⊗
7'⊗
31x

SURCHARGE STORAGE ROUTING SUPPLEMENT

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Qp_2"

b. Avg "$STOR_1$" and "$STOR_2$" and Compute "Qp_3".

c. If Surcharge Height for Qp_3 and "$STOR_{AVG}$" agree O.K. If Not:

STEP 4: a. Determine Surcharge Height and "$STOR_3$" To Pass "Qp_3"

b. Avg. "Old $STOR_{AVG}$" and "$STOR_3$" and Compute "Qp_4"

c. Surcharge Height for Qp_4 and "New $STOR_{Avg}$" should Agree closely

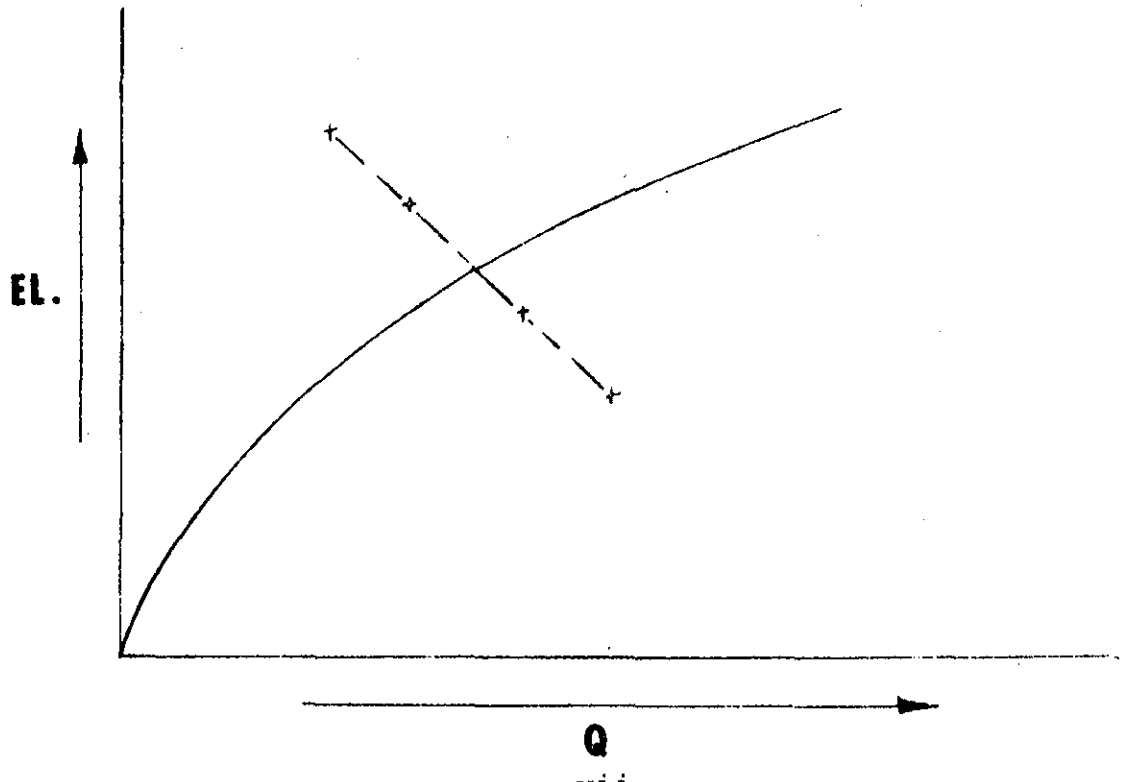
SURCHARGE STORAGE ROUTING ALTERNATE

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR}{19}\right)$$

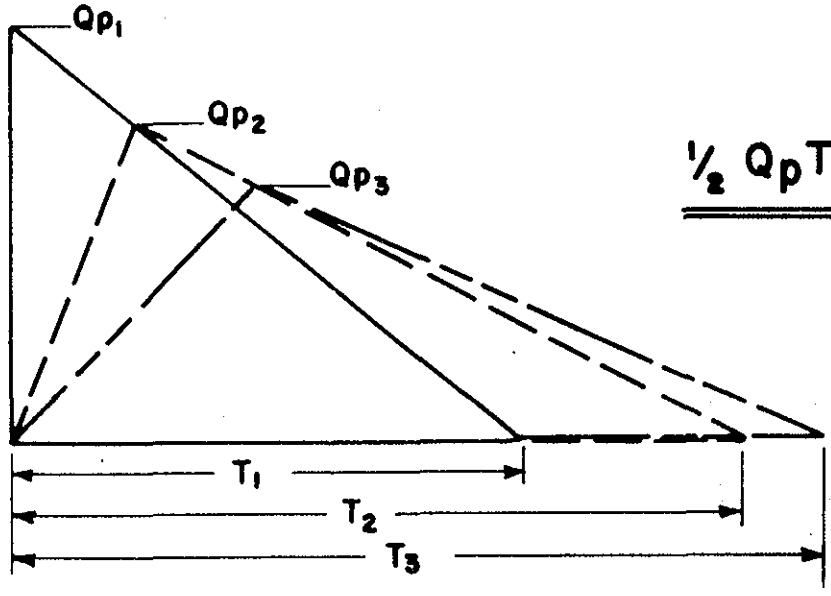
$$Q_{p2} = Q_{p1} - Q_{p1}\left(\frac{STOR}{19}\right)$$

FOR KNOWN Q_{p1} AND 19" R.O.

Q_{p2}	STOR	EL.
=====	=====	=====



"RULE OF THUMB" GUIDANCE FOR ESTIMATING DOWNSTREAM DAM FAILURE HYDROGRAPHS



$\frac{1}{2} Q_p T = 12 S$

STEP 1: DETERMINE OR ESTIMATE RESERVOIR STORAGE (S) IN AC-FT AT TIME OF FAILURE.

STEP 2: DETERMINE PEAK FAILURE OUTFLOW (Q_{p1}).

$$Qp_1 = \frac{8}{27} W_b \sqrt{g} Y_o^{\frac{3}{2}}$$

W_b = BREACH WIDTH - SUGGEST VALUE NOT GREATER THAN 40% OF DAM LENGTH ACROSS RIVER AT MID HEIGHT.

Y_o = TOTAL HEIGHT FROM RIVER BED TO POOL LEVEL AT FAILURE.

STEP 3: USING USGS TOPO OR OTHER DATA, DEVELOP REPRESENTATIVE STAGE-DISCHARGE RATING FOR SELECTED DOWNSTREAM RIVER REACH.

STEP 4: ESTIMATE REACH OUTFLOW (Q_{p2}) USING FOLLOWING ITERATION.

A. APPLY Q_{p1} TO STAGE RATING, DETERMINE STAGE AND ACCOPMANYING VOLUME (V_1) IN REACH IN AC-FT. (NOTE: IF V_1 EXCEEDS 1/2 OF S, SELECT SHORTER REACH.)

B. DETERMINE TRIAL Q_{p2}.

$$Qp_2(TRIAL) = Qp_1 \left(1 - \frac{V_1}{S}\right)$$

C. COMPUTE V_2 USING Q_{p2} (TRIAL).

D. AVERAGE V_1 AND V_2 AND COMPUTE Q_{p2}.

$$Qp_2 = Qp_1 \left(1 - \frac{V_{AVG}}{S}\right)$$

STEP 5: FOR SUCCEEDING REACHES REPEAT STEPS 3 AND 4.

APRIL 1978

APPENDIX E

INFORMATION AS CONTAINED IN THE NATIONAL INVENTORY OF DAMS